THE UNIVERSITY OF ALBERTA

MODEL STUDIES OF SCOUR AROUND BRIDGE PIERS AND STONE APRONS

A THESIS

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by

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ABSTRACT

Certain aspects of the problem of scour around bridge piers were investigated, as well as protection of the river bed against scour by means of loose stone aprons. Thus experiments were conducted for studying the effect on scour of the pier shape, length of pier, width of pier, restriction of flow due to the presence of the pier, angle of attack, debris collected on the nose, and flow characteristics. In the apron studies size and shape of apron, size of apron stone, as well as placement of aprons were investigated.

The entire thesis is the result of laboratory investigation referring to gravel rivers.

Suggestions for further investigation are given and the need of field measurements pointed out.

ACKNOWLEDGEMENTS

The author wishes to express his sincere appreciation to Dr. T. Blench for his valuable advice, constructive criticism, and painstaking work as editor; Professor A. W. Peterson for his valuable assistance in model construction and experimental procedures; the Alberta Research Council, the Department of Highways, and the University of Alberta for sponsorship under the Joint Highway Research Program; the National Research Council for providing funds for material and equipment; graduate students, and personnel of the Hydraulics Laboratory for the manual help they kindly gave the author.

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CHAPTER I. GENERAL

PREFACE

1.1. As early as 1873, Durant and Claye of France attempted to study the problem of river bed scour around bridge piers, (Ref. 4). Actually, the problem has existed since man first placed piers and abutments in rivers in order to cross larger streams. Since then, many a failure of an expensive structure has occurred, and been recorded, from this cause. A long epic poem tells the story of the Arta Bridge in Greece, built in the 1860's. Despite all efforts by the forty-five master masons and the large number of workers using all their skill in building the structure, the bridge would "sink in the winter". Finally, they got along by building a one span arched structure without piers. This structure still stands. The failures are attributed to the fast running mountain stream scouring the river bed around the piers and washing away the earlier bridges (Ref. 7).

1.2. The problem must have been accentuated in early days when massive stone piers had to be built, with relatively shallow footings, to support the large deadweight of the structure. Modern material, methods and equipment allow today's engineers to build slim, strong, streamlined piers with footings going deep into the river bed - in some cases until

they reach solid rock. In the case of the foundation of a pier on rock, the problem of scour around it is irrelevant. This is not always the case. The engineer designing a bridge will find himself concerned with the economic problem of how deep he will put his foundation in order to build a safe and economic structure. Still we have bridge failures attributed to the scour of the river bed around the pier footing.

1.3. All kinds of structures are now built on river beds and even yet our knowledge regarding scour is far from complete. Not even rules of thumb are generally accepted for prediction of scour, among engineers. This lack of design criteria has its roots mainly in the incomplete knowledge of mobile boundary hydraulics. Further, the specific problem of gravel rivers seems to have attracted less attention.

1.4. The study of the problem is a subject of mobile boundary hydraulics. As such, it has an almost infinite capacity. Every stream has its own personality. Characteristics such as bed material, side soil, suspended material and bed load, slope, velocity, cross section, etc. do not only vary from stream to stream but also there are great variations at different reaches of the same stream. In addition, the shape of flood hydrograph is another factor with individuality, even at different locations of the same stream. Therefor, an infinite number of cases of scour around bridge piers

exists. One solution, probably the best, is to build a model for the individual problem when it arises, providing that good records of floods have been kept, as well as records of sediment transport, at various stages. This is not economical though and, despite its merits, only in a few special cases involving high cost of construction is it done.

1.5. In cases where the expense to build a model is not justified, or other reasons prevent it, the engineer has to delve in the bibliography for rules and data to predict scour. Because of the many variables involved and the scarcity of data, the prediction of scour depends heavily upon the designer's experience, knowledge of mobile boundary hydraulics, and luck.

1.6. When the scour has been predicted, and therefore the safe foundation depth, another problem arises. It is the question of whether the foundation has to be so deep as to provide safety against the scour, or whether it would be more economical to protect the river bed around the pier against scour, and build a cheaper foundation.

1.7. The economical comparison can be done only if one knows how to predict scour and how to protect the river bed against scouring.

1.8. As mentioned before, this twofold problem has not

attracted very much attention by investigators, especially in the case of gravel rivers. There are many questions on the subject to be answered. This thesis deals with some of them.

PREVIEW OF LITERATURE *

1.9. In the preface of this work the phenomenon of scour of river bed around bridge piers, together with its significance to the engineer, was outlined. It was mentioned that design data are scarce and not applicable in all the specific problems which a design engineer faces. The only relevant literature discovered was:

Reference 1. Dr. Thomas Blench: Regime Behaviour of Canals and Rivers. Butterworth Scientific Publications, London, 1957.

1.10. Discharge intensity q plotted against scour depth multiplied by cube root of bed factor ($d_s\ F_b^{1/3}$) shows a good relationship when plotted on log-log paper. Scour data used were from sand models to very large sand rivers (Ref.2) and concerned scour around bridge piers, guide banks and spur noses. Bed load charges were believed to be small.

Ratios of maximum scour depth to "zero flood depth" are given for designing protective apron, but not for the prediction of scour that might occur without the aprons. Further, general rules for protection by means of aprons are given.

* See also "Bibliography".

Reference 2. Sir C. C. Inglis: The Behaviour and control of Rivers and Canals (with the aid of models) Part II. Central Waterpower Irrigation and Navigation Research Station Poona, India. 1949.

1.11. Inglis in this book devoted one chapter to scour data and another to aprons. He begins with Lacey's work on the subject and an attempt to explain the mechanics of scouring effect. His data are drawn from relatively large model experiments and field observations. Model work and field observations of a specific case are presented (Hardinge) Bridge).

Reference 3. E. M. Laursen and Arth Toch: Scour Around Bridge Piers and Abutments. The Iowa State Highway Commission and the Bureau of Public Roads. 1956.

1.12. The authors present model research and some field measurements. The effect of the geometry of the pier was emphasized.

Reference 4. E. M. Laursen: Scour at Bridge Crossings. The Iowa State Highway Commission and the Bureau of Public Roads. 1958.

1.13. A more elaborate work than the previously mentioned, by the same author and Toch. A mathematical analysis is presented, attempting to correlate scour with flow formulas.

Reference 5. P. Andru: A Study of Scour at Obstructions in Non-Cohesive Bed. M.Sc. Thesis, University of Alberta (unpublished) 1956.

1.14. Model studies of scour in the vicinity of obstacles carried out at the University of Alberta. Results were plotted along with those available from references 2,3.

Reference 6. R. K. Deeprose: Studies with Small River Models, M.Sc. Thesis (2 volumes) University of Alberta (unpublished) 1959.

Furthermore, in the individual sections of this thesis, reference is made to similar or relative works, specifically. The following chapter discusses the object of this thesis.

OBJECT OF WORK

1.15. As already mentioned very little is known on the subject of scour around bridge piers and protection against scour. Therefore the field is open to research and any aspect of the subject can be studied without duplicating the work of others.

1.16. We elected to study matters being of immediate practical as well as basic value; that is, the size and placement of loose stone aprons around piers in gravel rivers to suit a practical range of flow conditions.

1.17. Attempting the above one cannot avoid going into the scour problem itself. Therefore studies of scour preceded studies of scour protection in this work.

1.18. Gravel, i.e. material which has a settlement velocity proportional to the square root of material size, was chosen as bed material because:

(i) there was reason to believe it gives at least qualitative results dissimilar to those for sand;

(ii) practically no experimenters have used it;

(iii) gravel rivers are important in Alberta;

(iv) the settlement law seems to permit deductions to be drawn in terms of the ratio of apron gravel size to bed material gravel size. Anything larger than 1 mm may be considered gravel although popular useage would probably call material up to about 1/8" "sand". One soil mechanics classification (p. 7. Ref.1) starts gravel at 2 mm.

ANALYSIS OF THE PROBLEM

1.19 The following analysis of the scour problem refers mainly to the scour around bridge piers, although it can easily be generalized to refer to other obstacles such as

spurs and guide bank noses, which have been completely ignored in this work but are presenting the scour problem as well.

There are two main parties involved in the problem; The Stream, and the Pier.

1.20. The Pier; Properties of the pier contribute quantitatively and qualitatively to the scour. The properties of the pier involved are:

(i) Shape of pier. (The horizontal sections may be different at different elevations)

(ii) Width of pier.

(iii) Length of pier.

(iv) Quality of the surface of pier.

1.21. The Stream; Properties of the stream which affect or might affect scour are:

(v) Discharge intensity approaching the pier.

(vi) Viscosity of water.

(vii) Suspended load.

(viii) Bed load.

(ix) Sieve analysis of bed and side material. (It can be entered as average size)

(x) Debris. (This includes all kinds of floating and submerged material, but not ice cover over the whole area.

(xi) Specific gravity of bed material and bed load material. (usually the same)

(xii) Gravitational field intensity.

(xiii) Alignment to the pier of mean direction of flow.

(xiv) Time. (The shape of flood hydrograph effect is indirectly included in time)

(xv) Degree of restriction of river, or the ratio width of pier to breadth of river in the vicinity of the pier. (In other words, size of pier. When the pier characteristics were mentioned it was assumed that an infinitely wide river was involved).

Furthermore, when protection of river bed against scour is studied, the following quantities enter into the picture:

(xvi) Size of apron material (or ratio to the average size of bed material)

(xvii) Shape of apron material.

(xviii) Uniformity of apron material.

(xix) Specific weight of apron material.

(xx) Plan view size of apron and position relative to pier.

(xxi) Thickness of apron.

(xxii) <u>Elevation of aprons relative to mean bed.</u>

1.22. Velocity of water, depth of approaching flow and water surface slope are often used as independent variables, but are dependent on the factors stated above; use of any one permits one of the variables listed above to be omitted. Actually, for convenience depth of flow has been used in this thesis. Fluid density was not entered as the only fluid concerned was water; suspended load and gravitational field intensity were listed.

In this laboratory investigation, the effect of the following quantities were chosen to be studied:

1.23. Studies of scour.

(i) Pier shape (for five basic shapes)

(ii) Width of pier (w)

(iii) Length of pier (l)

(v) Discharge intensity (q)

(x) Debris

(xiii) Alignment of the pier to mean direction of flow. (θ^o)

(xiv) Time (t)

(xv) Degree of restriction, or size of pier (w/B)

That is:

$$d_s = fn\ (\text{shape}, l, q, g, t, w, B, \theta) \quad \ldots\ldots\ldots\ldots\ldots (1.1)$$

which can be made non-dimensional in various ways of which the following is convenient:

$$\frac{d_s}{d_c = \sqrt[3]{q^2/g}} = \text{fn}\left(\text{shape},\ \frac{l}{w},\ \frac{t\sqrt{g d_c}}{d_c},\ \frac{w}{B},\ \frac{q}{\nu},\ \theta\right) \quad \ldots(1.2)$$

In this q/ν is unlikely to be effective physically if it exceeds a certain limit and w/B might prove to be unnecessary if q is calculated as $Q/(B-w)$ instead of Q/B. Debris was not entered into equation 1.1 as it is not a basic quantity. g and ν were entered in order to assist in making the variables non-dimensional although they were practically constant throughout the experiment.

1.24. Studies of aprons.

(xvi) Size of apron material.

(xvii) Shape of apron material.

(xviii) Uniformity of size of apron material.

(x) Elevation of apron relative to mean bed.

(xx) Plan view size of apron.

(xxi) Thickness of apron.

Having decided on the function to correlate, the next problem is the extent to which the observations can be scaled up to river size. This calls for a brief discussion of river models appearing next.

RIVER MODELS

1.25. Models of any description have been, and are, built for purposes of studying the performance of expensive prototypes. Models are even built to study the esthetic appear-

ance of the prototype, for example buildings, automobiles, ships, etc. If a model, built for esthetic study purposes only, can be misleading, then it is easy to imagine the difficulties that are encountered by investigators working with dynamic models in the study of complicated physical phenomena. Unwary workers are often misled.

1.26. The model is usually built with geometrical similarity to the prototype - seldom full scale. In addition to the geometry of the model there are other important quantities such as power, friction, etc. which have to be scaled. Physical laws allow a modelmaker to decide upon only a certain number of such ratios. Then the remainder are superimposed by nature's relationship to those the modelmaker decided upon. The abilities of the modelmaker are tested in the selection of the ratios which he will impose.

1.27. There are certain quantities which cannot be scaled. This is best illustrated in the classical example of the floating razor on the wash basin. It cannot be considered as a model of a one ton steel plate on a lake surface. Simply, the surface tension cannot be scaled at any other scale but unity using the same liquid at the same temperature.

1.28. Mobile boundary models present special problems to the modelmaker. Consider, for example, the case of a

10 ft. deep river with bed material in the sand range. Suppose that a model is built at 1:100 scale, and the modelmaker, convinced by other field's experiences, tries to maintain geometrical similarity. He would demand that the model run at a depth greater than 0.1 ft. so would have to use fine clay for bed. Clay will not be transported by flowing water except in suspension, and may cohere so as not to move at all, thus giving a different picture than in the prototype where the bed material does not cohere, and moves by rolling and saltating. Knowledge of mobile boundary hydraulics will assist the construction of a river model in so far as it provides laws of self-adjustment that are not available in rigid boundary hydraulics. These laws permit scaling without geometric similarity.

1.29. Still there will be quite a few sources of "scale effect" errors even in the best designed model. For example, the sliding angle of bed material cannot be scaled between model and prototype and therefore will cause an error in assessing scour depth and volume.

1.30. The obvious drawbacks of all models are accentuated in river models. This fact does not devaluate the importance of those models, of general or of specific nature. On the contrary, river models are very important and necessary, one reason being that phenomena in the prototype are very slow in

developing and may never happen in a man's lifetime (for example the 1000 year flood). Furthermore, measurements in the field are impossible for certain quantities.

1.31. The basis of all practical model scaling is to hold as many factors as possible the same in model and prototype respectively so that variable "quantities" are reduced to a minimum. In the present case a principal decision was to keep the settlement velocity law of the bed material the same in model and prototype (i.e. $V_s^2/gD_m = \text{cont}$) by using model material in the gravel range; for sand $V_s^2/gD_m = fn(D_m)$. With this precaution it seems that apron material size can be represented fairly as a multiple of bed material size.

DESCRIPTION OF THE EXPERIMENTAL SETUP

1.32. In order to carry out the previously mentioned task of studying certain aspects of the scour and protection against scour problem, the following setup was constructed in the New Hydraulics Laboratory of the University of Alberta (Fig.1).

1.33. A flume was constructed measuring 44 inches in width, 130 feet in length, and 24 inches in depth. (Fig. 1, 2, and 3) The walls of the flume, which were prefabricated from galvanized sheet metal in 10 feet interlocking sections,

were bolted on "Kindof" channel imbedded in the horizontal concrete floor. (Fig.3) The flume was then waterproofed with a lining of polythene plastic.

1.34. The head works occupied the first 10 feet of the flume. The flume walls were extended another 12 inches higher, bringing the total depth of the head works to 36 inches. Therefore the stilling basin, where the pump outlet discharged, had dimensions of 36 inches in height, 44 inches in width and 9 feet in length. The stilling basin was followed by an energy dissipator made of 42 leaves of corrugated sheet metal 12 inches long, 36 inches wide and 36 inches high. The corrugated leaves were welded vertical and parallel to the flow.) (Fig.1 and 3)

1.35. A sand trap occupied the last 5 feet of the flume. An adjustable weir held the sand at station 125 and another weir, regulated by means of a crank at station 130, maintained the desired tailwater level. (Fig. 1 and 2).

1.36. Therefore a working length of 115 feet was left for modeling a gravel river bed in the flume. On the side of the flume, stations were marked at 1 foot intervals from 0 to 130.

1.37. Gravel for the flume was sieved from natural sand pit material (river deposit at Devil's Lake, Alberta); this produced the size distribution shown in the "sieve analysis"

plot (Fig. 5). The specific weight of the material was 1.54 and the true specific weight was 2.58, therefore poroscity was 39.1%. The flume from station 10 to station 125 was filled initially with this gravel to an average depth of 11 inches, leaving the water to adjust the bed slope.

1.38. Manometers were tapped on the side of the flume, 8 inches above the floor and connected to a manometer bank. The manometer bank, inclined 30° to the horizontal, was graduated to read directly in hundredths of a vertical foot. The fourteen manometers were tapped at the following stations: 21, 35, 41, 47, 53, 66, 71, 77, 83, 89, 95, 101, 107, 121.

1.39. Steel pier stands (Fig. 3) were used to support the pier models at the following stations: 35, 47, 59, 71, 83, 95, 107. They were numbered from 1 to 7 correspondingly.

1.40. Temperature of water was measured by thermometer permanently installed in the flume.

1.41. Water supply to the flume was afforded by a vertical "Pamona" type Fairbanks Morse pump electric motor driven, and capable of delivering up to 14 cfs of water. The official characteristics are shown on Fig. 6. The supply line was 14 inches aluminum irrigation piping with a 14 inch gate valve near the outlet. Another 14 inches gate valve was in the 14 inch "return line" (or pressure relief line). Combination of openings of the two valves permitted regulation of discharge from zero up to the capacity of the pump which was about three

and one half times the maximum discharge employed. This arrangement ensured steadiness of discharge.

1.42. Measurement of the discharge was accomplished by an orifice type meter (Canadian Meter Co. Ser. No. DF6735) with an orifice disk of 7.030 inches bore, in the 14 inch line.

1.43. Sediment was fed by a locally made electric motor-driven sediment feeder of the pulsating type at predetermined rates. (Fig. 2). Sediment collected in the sand trap was dried and weighed for occasional checks that rate of sediment outflow had equalled rate of injection.

1.44. A graduated glass was employed for measuring scour volume. The graduations were at one-half cubic inch intervals, from zero to fifty cubic inches. The glass was used as described in experimental procedures.

1.45. Scour depth was measured by a ruler of about one-fifth of a mm thickness, graduated in tenths of an inch.

1.46. A device, working on the pantograph principle, was introduced for taking scour contours; this was an innovation over the methods employed previously, i.e. marking contours with sugar using a cake decorator as the water table was receding, or using colored layers of sand and photographing the result. This device actually consisted of a pantograph mounted on a draughting board. The pivot pin was fastened

on the edge of the board and the pointer was replaced by a longer adjustable one. Thus the point could be adjusted to draw the intersection of the scour surface and the river bed. The resulting curve was marked zero. Then the pointer would be elongated by one inch, and the minus one inch contour would be drawn by simply following the scour surface with the end of the pointer. In the same way all contours were drawn at the desired scale. The results were better than one could expect from the previously employed methods, but they could be excellent with a precision-made instrument. A suspended pantograph should be modified for the purpose.

Experimental procedures, as followed, are described in the following chapter. Any deviation from this general procedure is mentioned in the relative chapter.

EXPERIMENTAL PROCEDURE

In the previous section the setup and equipment employed in the experiment were described. This section outlines procedure common to all experiments.

1.47. The flume, initially, was filled with gravel (Fig. 5) to a depth of 11 inches. Then a discharge of 1 cfs, to represent a dominant flood condition, was introduced together with a charge of 1.7 parts per hundred thousand by weight. This was run for a total time of 150 hours to

create regime conditions. Actually, after the first day, in-regime conditions were attained as judged by the rate at which material collected in the sand trap equalled the rate of charge injection.

1.48. In advance of each individual experiment, the model piers were placed at the stands as described in the corresponding chapters and illustrations. Then the discharge of 1 cfs was run for times varying between 16 to 24 hours, far in excess of what was needed to readjust the flume to the original regime conditions and wipe out the discrepancies of the bed after the pier placement. It was observed that the presence of the piers caused no appreciable deviation from regime conditions. This might have been expected from calculating the difference of water depth upstream and downstream of a model pier by the Rehbok formula, although it is not supposed to be accurate when it is applied to such small depths.

The formula (Ref. 8) is:

$$h = K \frac{w}{b-w} \cdot \frac{V^2}{2g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1.3)$$

One of the worst cases encountered in the flume, the case of the 3" pier with a discharge of 3.6 cfs is used in this example. Here $w = 3''$, $b - w = 44-3 = 41''$ $V = 1.89$ f/s and $K = 1.30$ for rounded nose piers, or

$$h = 1.30 \cdot \frac{3}{41} \cdot \frac{1.89^2}{2g} = 0.00517 \text{ ft.}$$

which was negligible.

1.49. When regime conditions were attained, the discharge was stopped and the flume drained. The moment the pump was switched off the supply line started siphoning water from the flume stilling basin to the sump; thus the flume was drained from both ends. This modified the surge wave so as not to disturb the bed. Then, the scour was filled with gravel of the same material (or aprons were laid) and the flow started again at a slow rate of increase up to 1 cfs. Above the 1 cfs and to the final value of the discharge run in the experiment, the discharge was raised at intervals of 1/4 hours at 0.15 cfs. (Fig. 7). In most of the experiments the discharge was not raised above 2 cfs because dunes started appearing at this stage, making measurements difficult. As demonstrated by experiment VII, where the discharge was raised up to 3.60 cfs, the scour depth had reached quite a high percentage, of its 3.60 cfs value, at the stage of 2 cfs.

1.50. The water surface was kept parallel to bed by observation of the manometer bank and regulation of tailwater by the adjustable weir.

1.51. Water depth was measured by a thin (1/5 mm) ruler graduated in tenths of an inch, and having a small footing

so it would not sink into the gravel. Eight such direct measurements were made at two sections (four at each) and averaged. The two sections were 1 ft. apart. Another measurement of depth was obtained from the manometers. The manometer bank was inclined at 30° to the horizontal and graduated in such a manner as to read directly in tenths of a vertical foot. The manometers were sucked by aspirator until no air was left in the plastic transparent tubing connecting manometer tap to the glass tube in the bank.

1.52. Scour depth was measured by rule from the lower point of the scour surface to the surrounding bed. Then the water depth was added to it to produce "scoured depth" for analysis.

1.53. In Experiment VII a plastic transparent hollow pier model was used, as described in Section 2.43. The pier was graduated in inches and quarter of inches. Reading the scour depth below the initial bed, was accomplished by using a mirror, such as a dentist uses.

1.54. Contours of the scour surface were taken by the contourgraph, Section 1.46. A sheet of paper was fastened on the board and the pointer adjusted to the average bed elevation in the vicinity of the pier. Then the intersection scour surface - flume bed - would be followed by

the pointer and the result appears in the Figures 14, 18 and 23 marked "zero". Then the pointer would be elongated by one inch and the scour surface would be followed and the -1" contour result, and so forth. The procedure is comparable to the one followed when photogrammetric equipment are used in obtaining land contours from 3-D photographs.

1.55. Generally, the experimental work was conducted in such a manner as to keep all the variables fixed except the one under study in the individual experiment. This method of studying separately each variable allows the magnitude of the variables effect to be fully appreciated without doubts of interference.

DIFFERENCES BETWEEN GRAVEL AND SAND BEHAVIOUR

1.56. The modes of transportation of solid material by flowing water are (i) in suspension and (ii) as bed load. Relatively fine materials move in suspension. Bed load is the material moving along the bed "rolling" and "saltating". If there is no bed movement, the bed is said to be "inactive" - in contrast with a "moving" or "active" bed.

1.57. In an active sand bed "dunes" are formed and travel downstream as long as the water velocity remains

below critical. As the critical velocity is attained, the "dunes" flatten giving way to another mode of bed movement known as sheet flow. Above critical water velocity, "anti-dunes" appear and travel upstream. See also Section 1.31 and Reference 1.

1.58. When the flume described in the chapter "experimental setup" was first run the peculiarity of "sheet bed movement" at low velocities was observed. Starting with an inactive bed, the velocity of the water is gradually increased by either imposing a steeper water surface slope or increasing the discharge. At some velocity, particles of bed material start rolling. As the velocity increases further, the number of moving particles increases until the whole of the bed is moving in a flat "sheet" flow manner. Further increase in velocity above a second limit results in changing of this mode of bed transport to dune formation. Dunes are formed and travel at slow speeds downstream. These dunes are of elongated shape of a depth varying up to 1 inch and wave length varying from 10 inches up to 30 inches.

1.59. At any stage, steady transportation continues as long as sediment is fed at the rate it is moving. If the rate of sediment supply is decreased, the flow will pick up bed material and the bed will tend to degrade to a milder slope where there will be no pick-up. With the gravel, if

the bed movement is in dunes and suddenly the sediment feeding is stopped while the discharge remains constant, the bed will continue to move for some time in dunes and the bed will degrade; but sooner or later sheet flow will occur with only a few particles moving and the bed will become inactive eventually and show no trace of dunes.

1.60. This is in contrast to the aforementioned modes of sand bed movement where dunes form as soon as bed movement starts and remain after it has stopped. This phenomenon is mentioned, without conviction, by Albertson in a recent paper. (Ref. 10). Whether the phenomenon is related to settlement velocity, or relative size of sediment, or Reynold's Number combined with them, will have to be decided by a separate investigation.

LEGEND OF FIGURE I

1. Pump.
2. 14" Return Line.
3. Flow Meter.
4. 14" Supply Line
5. Stilling Basin
6. Energy Dissipator
7. Sand Feeder.
8. Testing Part of Flume.
9. Manometer Bank.
10. Sand Trap.
11. Sharp Crested Weir
12. Stilling Well

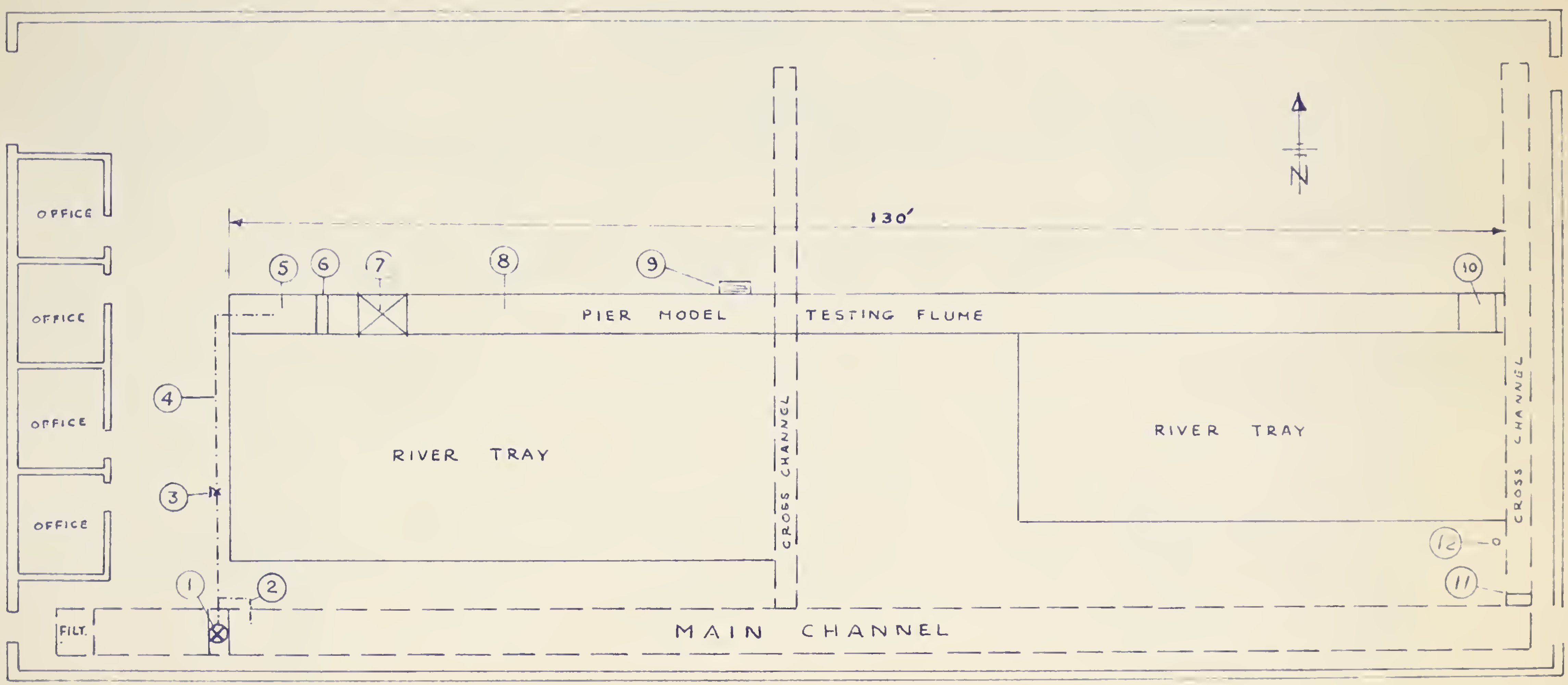

FIG 1. HYDRAULICS LABORATORY PLAN

Scale: 1/16" = 1'

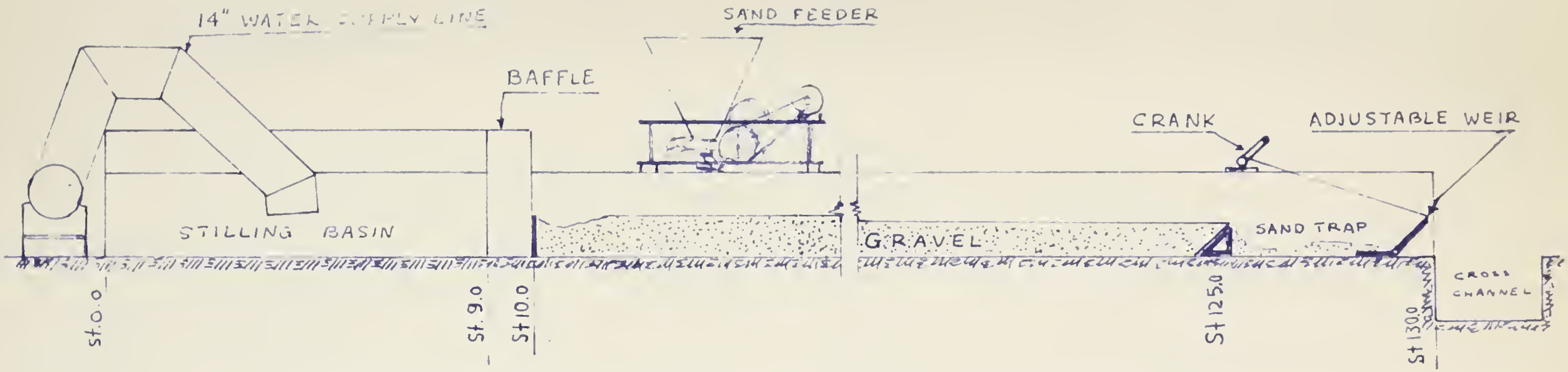

FIG. 2. SECTION OF FLUME ALONG ITS AXIS.

SCALE 1/4" = 1'

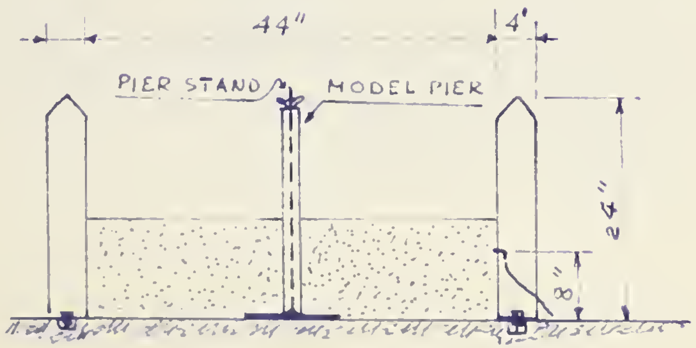

FIG 3. FLUME CROSS SECTION

SCALE 1/2" = 1'

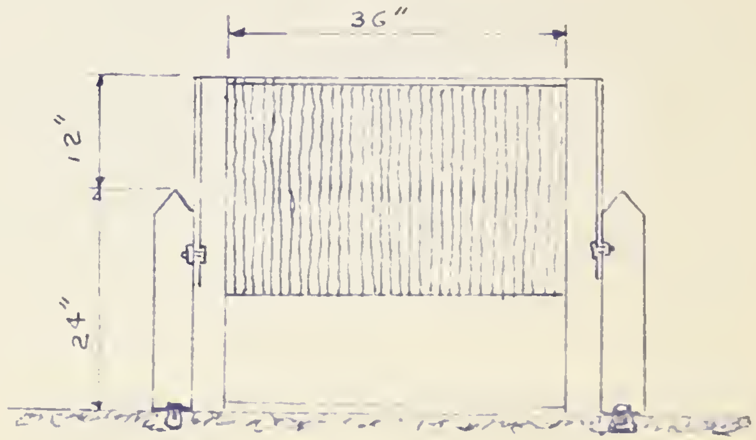

FIG 4. STILLING BASIN CROSS SECTION

SCALE. 1/2" = 1'

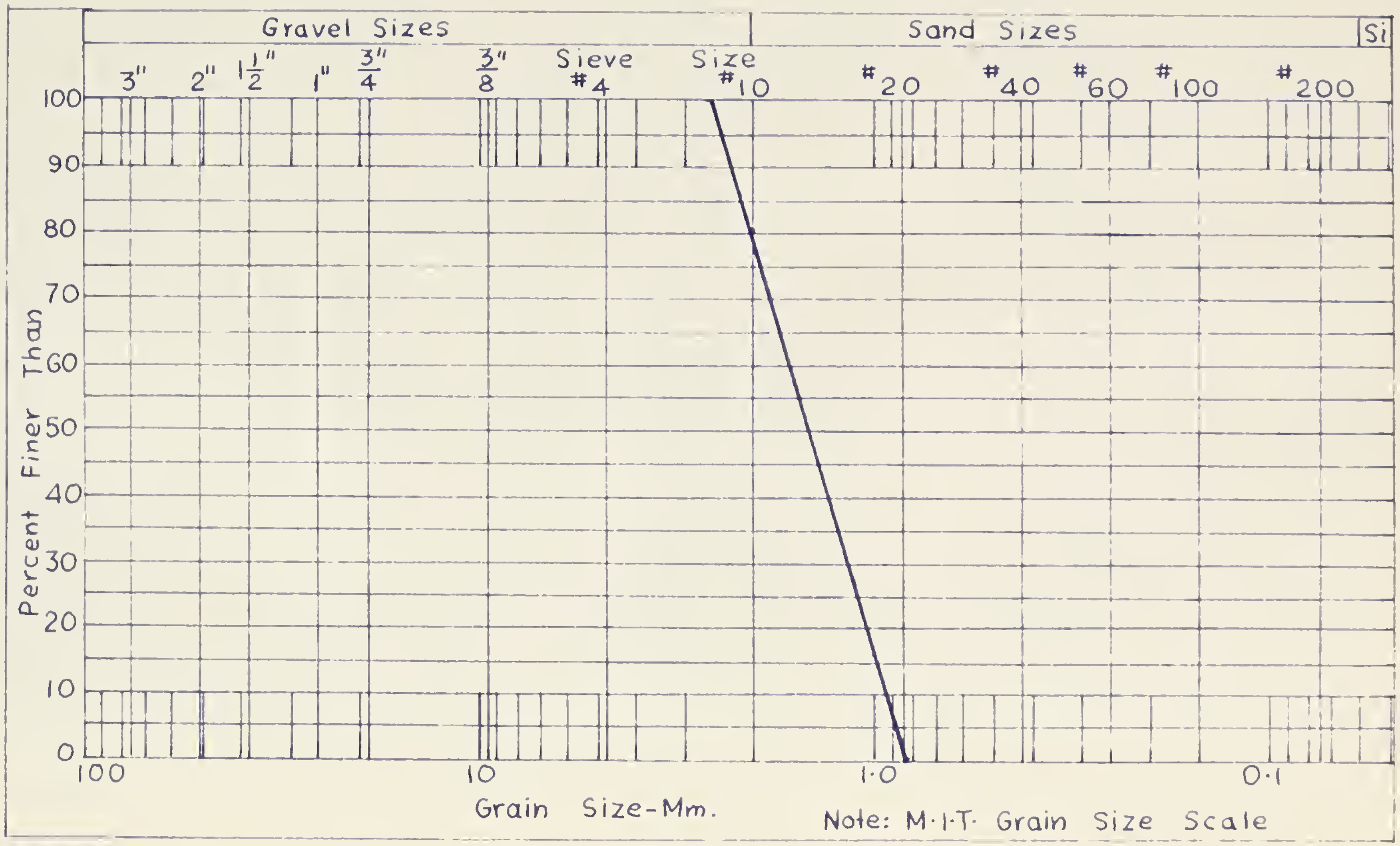

FIG 5. SIEVE ANALYSIS OF FLUME MATERIAL

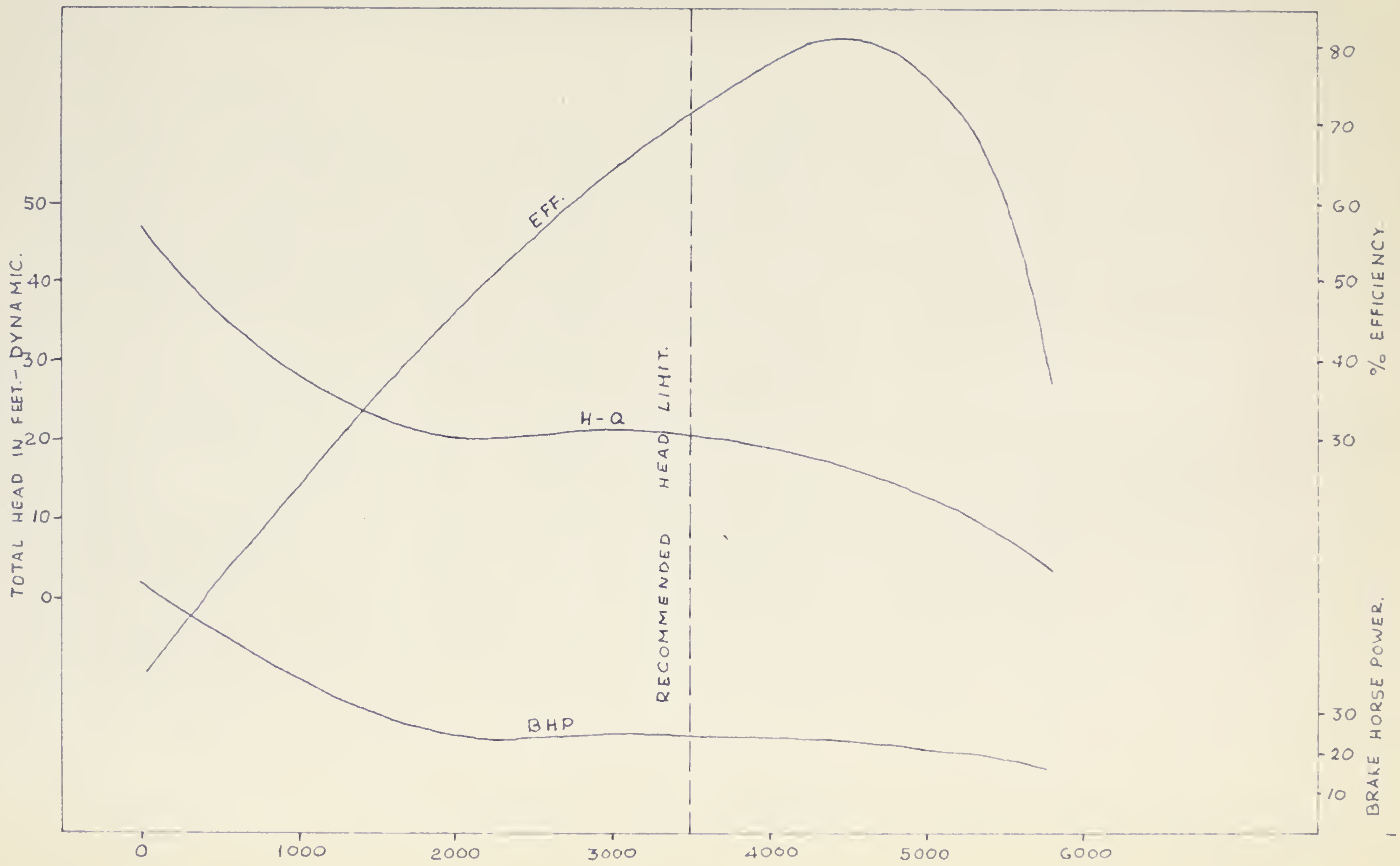

FIG 6 OFFICIAL TEST CHARACTERISTICS OF PUMP (FAIRBANKS, MORSE & CO curve No 14NXRSN)

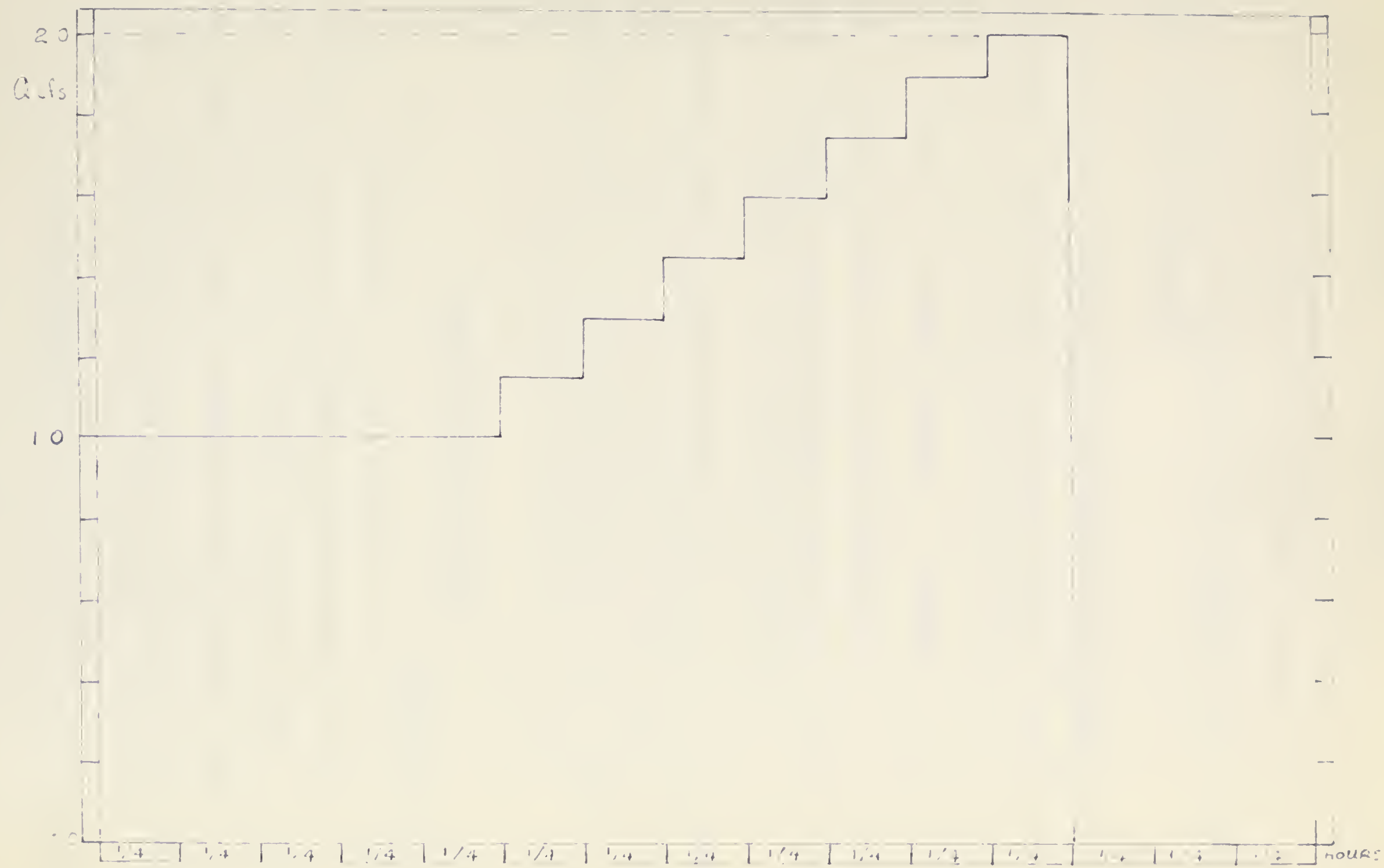

FIG 7. RATE OF INCREASE OF DISCHARGE (EXPERIMENTS I, II, III, IV V VI and VIII).

CHAPTER 2 SCOUR STUDIES

2.1. In this chapter the experiments conducted to study scour are presented, analyzed, and compared with similar work, if any available.

2.2. Scour is caused when flowing water picks up and transports material from a certain place at a different rate than it does in the surrounding area of this place. This can result from the presence of eddies or increase in flow velocity locally, or their combination. For the different theories on how transport starts and proceeds, the reader is referred to Dr. Blench's book (Ref. 1), to Dr. Karakassoni's lecture notes (Ref. 7) and S. Leliavsky (Ref.9). It would be a deviation from the purpose of this practical work to present those theories here, some of which are of only academic or historical interest.

EFFECT OF PIER SHAPE

(Experiment 1)

2.3. As mentioned in the analysis of the scour problem the shape of pier must affect the magnitude of scour. In this experiment only this effect was under study; all other quantities were kept the same for all the pier models tested. Therefore the equation 1.1 (section 1.23) can be written:

$$d_s = \text{fn (shape)} \qquad (2.1)$$

2.4. Accordingly, pier shapes were modelled as described in Table 1 and illustrated in Fig. 8.

	Model	Shape	Overall size	Stand*
(i)	P1	Lenticular	1 x 6 inches	2
(ii)	P2	Elliptical	1 x 6 "	3
(iii)	P3	Rounded nose	1 x 6 "	5
(iv)	P4	Beveled nose	1 x 6 "	4
(v)	P5	Rectangular	1 x 6 "	6

* For location of stands please see "experimental setup". (Section 1.39)

Table 1 - Model piers of Experiment I

2.5. After placement of piers, the flume was brought up to regime conditions (Sec. 1.49.). Then the discharge was raised up to 2 cfs at the rate shown in Fig. 7. Measurements of flow characteristics were not taken, as the same conditions were prevailing throughout the flume and relative scour was sought.

2.6. The scour developed around these slender models was not of great magnitude, but the considerable effect of pier shape on scour was demonstrated. The quantitative results are as one could expect from knowledge of fluid mechanics. The lenticular and elliptical models yielded the lesser scour. Despite this, they are impractical mainly due to high construction expenses involved and therefore their

[illegible]

[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]

[illegible]

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[illegible]

testing was of rather academic interest. The rectangular section, cheapest of all to build, was abandoned by even earlier bridge builders, but still there may be cases where the results could be useful. The rounded nose and bevelled nose shapes are most popular in today's construction. They affect scour of about the same magnitude. The rounded nose shape is in favor, over the cheaper bevelled, as it appears to collect less debris and its aesthetic appearance is good.

2.7. Similar tests were conducted by Laursen and Toch (Ref. 4) who mention that the M.Sc. thesis of D. E. Schneible was devoted to this topic. Their model pier's length to width ratios were 2 and 3 while in this experiment it was 6. Also, Laursen does not mention testing a bevelled nose model. Their results are consistent with the author's except in the case of rectangular pier model. According to Laursen and Toch, the rate of scour (d_s) of a rectangular pier to the same of rounded nose pier (d'_{sR}), (measured from average bed) is: $\frac{d'_s}{d'_{sR}} \doteq 1.10$, while the author found: $\frac{d'_s}{d'_{sR}} \doteq 1.67$.

2.8. Table 2 has been devised to show results clearly and should be read with Figure 8 and Table 1.

Column 1 shows the models tested and correlates Table 2 with Table 1.

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

Column 2 shows the angle of attack (θ^{o}).

Column 3 shows the shape of the models.

Column 4 shows the overall size of the models.

Column 5 shows the square area of the cross section of the models (a).

Column 6 shows the perimeter of the cross section of the model (L).

Column 7 shows the maximum scour depth measured from bed to deepest point of scour surface (d_s').

Column 8 shows depth of flow (d)

Column 9 shows maximum scoured depth, measured from water surface; i.e. $d_s = d_s' + d$

Column 10 shows scour volume V_s.

Column 11 shows ratios of maximum scoured depth of each of the models to same of the rounded nose model.

Column 12 shows ratios of scour volume of each of the models to same of rounded nose model.

2.9. Depth from water surface to the deepest point of scour surface was used in this work since this depth is most likely to have simple dynamical significance. However, depth of scour below original bed emphasizes the effect of shape more clearly.

2.10. $\frac{d_s}{d_{sR}}$ is likely to be more interesting to a bridge engineer who enquires "what do I gain, in general, by streamlining noses?" (The rounded nose pier was taken as standard in this thesis).

2.11. $\frac{V_s}{V_{sR}}$ gives a rough estimate of the relative volumes of the stone apron likely to be required for different designs.

Model	θ^o	Shape	Overall Size in.	a sq.in	L in.	d'_s in.	d in.	d_s in.	V_s c.in.	$\frac{d_s}{d_{sR}}$ *	$\frac{V_s}{V_{sR}}$ *
P1	0	Lenticular	1 x 6	3.10	12.30	1.20	4.20	5.40	10	0.948	0.40
P2	0	Elliptical	1 x 6	4.71	12.45	1.25	4.20	5.45	14	0.955	0.56
P3	0	Rounded nose	1 x 6	5.78	13.14	1.50	4.20	5.70	25	1.000	1.00
P4	0	Beveled nose	1 x 6	5.00	12.48	1.60	4.20	5.80	25	1.018	1.00
P5	0	Rectangular	1 x 6	6.00	14.00	2.65	4.20	6.85	30	1.027	1.20
1	2	3	4	5	6	7	8	9	10	11	12

* R refers to the rounded nose model.

Table 2 Results of Experiment I

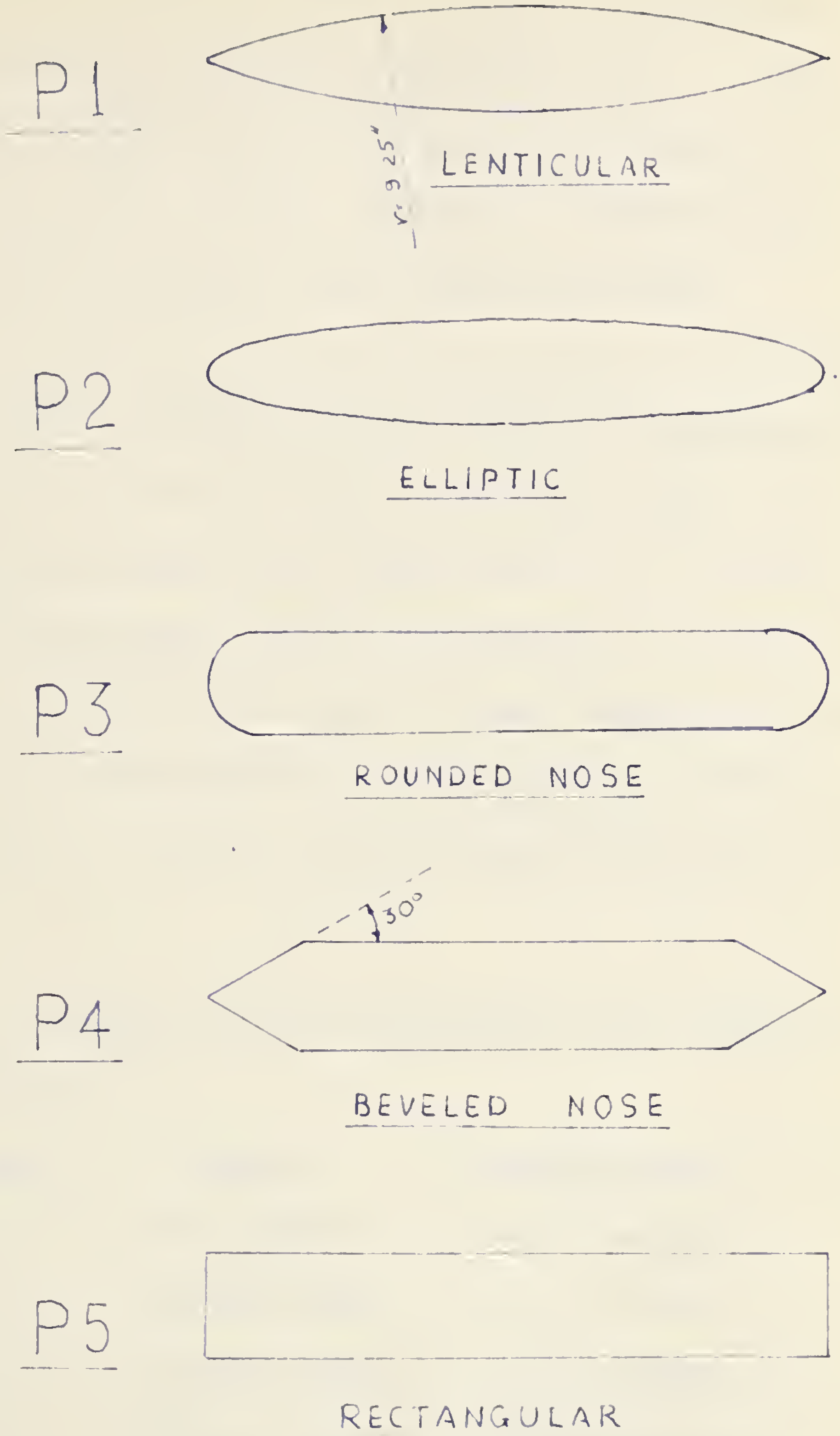


FIG 8. MODEL PIERS OF EXPERIMENT I

EFFECT OF VARIATION OF PIER LENGTH

(Experiment II)

2.12. The object of this experiment was to study the effect of pier length variation on scour formation as mentioned in the scour problem analysis. Therefore the function (1.1) of section 1.23) can be written:

$$d_s = fn\ (l) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.2)$$

Today's methods and materials of construction allow engineers to build slim streamlined piers but there is an increasing demand for longer piers for multiple lane highways, or multitrack railroad bridges, as well as their combinations. No previous work on the subject is available, therefore this experiment was considered in order to evaluate the effect on scour of pier length variation, if it has any effect.

2.13. Pier models of this experiment are described in the following Table 3.

	Model	Shape	Overall Size	Stand
(i)	P 6	Round shaft	1 x 1 inches	1
(ii)	P 7	Rounded nose	1 x 2 inches	2
(iii)	P 8	Rounded nose	1 x 5 inches	3
(iv)	P 9	Rounded nose	1 x 10 inches	4
(v)	P 10	Rounded nose	1 x 15 inches	5
(vi)	P 11	Rounded nose	1 x 20 inches	6

Table 3 - Model piers of experiment II

The experimental procedure followed is the same as in experiment I (Sec. 2.5).

2.14. The results of this experiment are not as spectacular as one might expect. They appear on Table 4 and Figures 9 and 10. In Figure 9 the scour depth versus length of pier was plotted non-dimensionally, that is d_s/d_c against l/d_c. The points scatter about a horizontal line showing that the affect of pier length on maximum scour depth is not significant, at least after a certain value of length to width ratio.

In Figure 10 volume of scour versus length of pier is plotted. V_s is the volume of scour while V_p is the volume of pier per linear (vertical) unit of length; i.e. the cross-sectional area of pier. So the ratio is a mean depth of scour based on pier area which might help an engineer to visualize its amount.

2.15. A satisfactory explanation of the peculiarity of the curves of both Fig. 9 and Fig. 10 is not found. Most probably, in the writer's opinion, the flow established a regime on either side of a long pier with lessening effect of the downstream nose. It can be said that a long pier is a flow divider rather than an obstruction.

2.16. Table 4 should be read jointly with table 3 and Figures 9 and 10. For explanation of table 4 the reader is referred to section 2.8.

Model	Overall size	a sq.in.	L in.	d'_s in.	d in.	d_s in.	V_s c.in.	d_c in.	$\frac{d_s}{d_c}$	$\frac{V_s}{V_p}$
P6	1 x 1	0.78	3.14	1.50	4.20	5.70	5	2.51	2.26	6.4
P7	1 x 2	1.78	5.14	1.25	4.20	5.45	8	2.51	2.17	4.5
P8	1 x 5	4.78	11.14	1.30	4.20	5.50	17	2.51	2.18	3.5
P9	1 x 10	9.78	21.14	1.50	4.20	5.70	23	2.51	2.26	2.3
P10	1 x 15	14.78	31.14	1.40	4.20	5.60	13	2.51	2.21	0.9
P11	1 x 20	19.78	41.14	1.30	4.20	5.50	10	2.51	2.18	0.5
1	2	3	4	5	6	7	8	9	10	11

N.B. Pier models were aligned with mean direction of flow.

Table 4. Results of Experiment II

FIG 9 EFFECT OF PIER LENGTH ON SCOURED DEPTH.

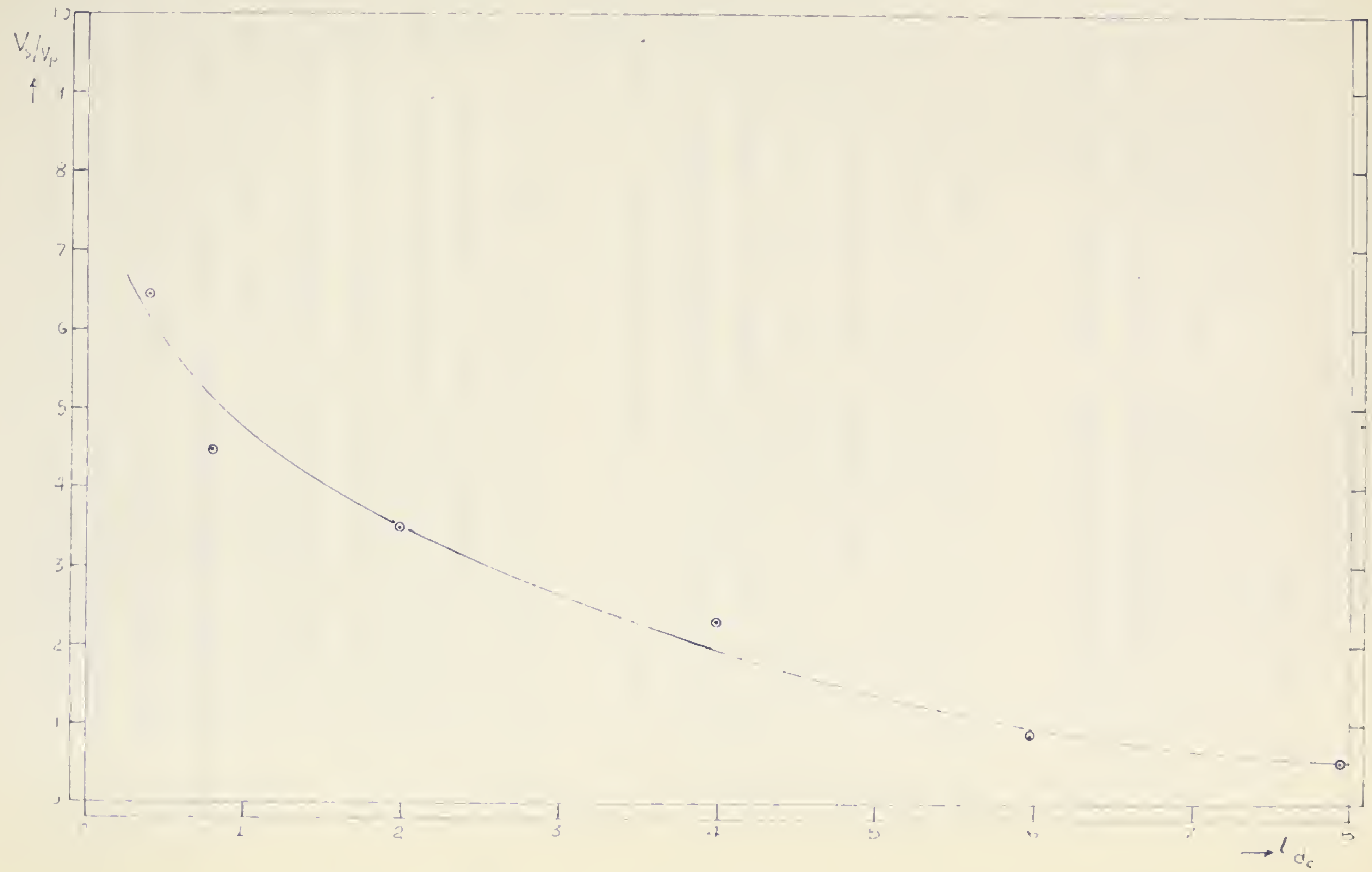


FIG 10 EFFECT OF PIER LENGTH ON VOLUME OF SCOUR

EFFECT OF PIER WIDTH VARIATION

(Experiment III)

2.17. The object of this experiment was to study the magnitude, if any, to which scour is affected by pier width variation. Therefore, equation 1.1 of section 1.23 can be written.

$$d_s = fn(w) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.3)$$

Actually the experiment should have been conducted in such a setup as to eliminate flow contraction. A proper flume should have been either very broad, or of varying width. In this experiment the standard width flume, as described in section 1.33, was employed and of course contraction of flow existed. Therefore the above equation should be replaced by:

$$d_s = fn \frac{w}{B} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.4)$$

2.18. The magnitude of flow contraction varied for the different models tested from 2.3 up to 9.1% In section 1.48 the backwater effect of a model pier contracting the flow 9.10% at a discharge of 3.6 cfs was found to be negligible (in this experiment only 2 cfs was run). Also Laursen and Toch (Ref. 4) state that flow contraction up to 10% has no appreciable affect on scour. If the above is to be believed then it can be said that the experiment yielded results on pier width variation and equation 2.3 of section

2.18 holds.

2.19. Model piers employed in this experiment are described in Table 5, below, and illustrated in Fig. 11.

	Model	Shape	Overall size	Stand
(i)	P3	Rounded nose	1.0x6.0 inches	1
(ii)	P12	" "	1.5x6.0 "	2
(iii)	P13	" "	2.0x6.0 "	3
(iv)	P14	" "	2.5x6.0 "	4
(v)	P15	" "	3.0x6.0 "	5
(vi)	P16	" "	3.5x6.0 "	6
(vii)	P17	" "	4.0x6.0 "	7

Table 5. Model piers of Experiment III

2.20. The experimental procedure followed was the same as for the previous experiments I and II (Sect.2.5).

2.21. The results of this experiment are clearly presented in Table 6 which should be read jointly with Table 5 and Figures 12, 13, 14 and 15 a, b, c, d, e, f, g. In Fig.12 scoured depth versis width of pier is plotted non-dimensionally, that is d_s/d_c against w/d_c. Further, in Figure 13 scoured depth is plotted against flow intensity, that is d_s/d_c versus q'/q where q' and q are discharge intensities in the restricted and unrestricted section respectively. This

later plot is for the reader who believes in the significance of flow contraction effect in this experiment. The author believes that results represent the effect of pier width as explained in section 2.19 and as it is further supported by the fact that maximum scour occurs in front of the upstream nose of the pier as illustrated in Figures 15a, b, c, d, e, f, g. If the affect of flow contraction is to be studied properly, a fixed size pier should be used in varying width flume. In Figure 14, scour volume is plotted against cross-sectional area of pier model. For discussion of this plotting, the reader is referred to section 2.14. Figures 15a, b, c, d, e, f, g show the shape of the scour surface produced around the different pier models of this experiment.

2.22. Dr. T. Blench (Ref.1) and Sir C.C. Inglis (Ref.2) appreciate the pier width as scour affecting factor. Laursen and Toch integrate this factor with flow contraction. Another similar experiment appears next.

Model	0	a sq.in	L in.	d'_s in.	d in.	d_s in.	V_s c.in.	$\frac{w}{d_c}$	$\frac{d_s}{d_c}$	$\frac{q'}{q}$	$\frac{V_s}{V_p}$
P3	0	5.78	13.14	1.48	4.20	5.68	20	0.40	2.26	1.025	0.35
P12	0	8.52	13.71	2.48	4.20	6.68	63	0.60	2.66	1.037	7.40
P13	0	11.14	14.28	3.12	4.20	7.32	118	0.79	2.92	1.050	10.60
P14	0	13.66	14.85	3.68	4.20	7.88	171	0.99	3.13	1.062	12.53
P15	0	16.06	15.41	4.50	4.20	8.70	230	1.20	3.45	1.075	14.30
P16	0	18.65	16.00	4.75	4.20	8.92	300	1.40	3.55	1.087	16.10
P17	0	20.57	16.57	5.00	4.20	9.20	375	1.60	3.66	1.100	18.25
1	2	3	4	5	6	7	8	9	10	11	12

Table 6. Results of Experiment III

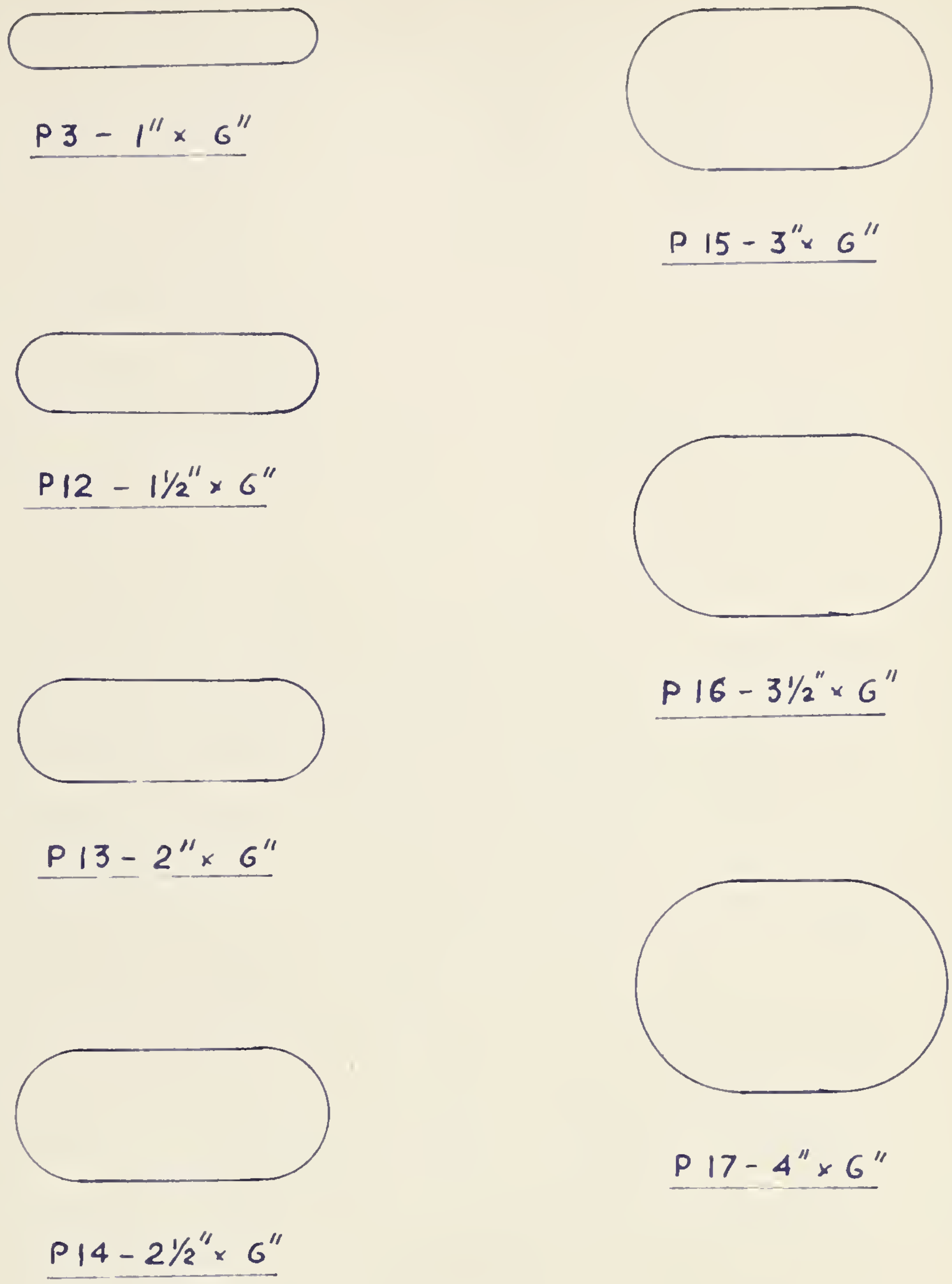


FIG. 11. MODEL PIERS OF EXPERIMENT III

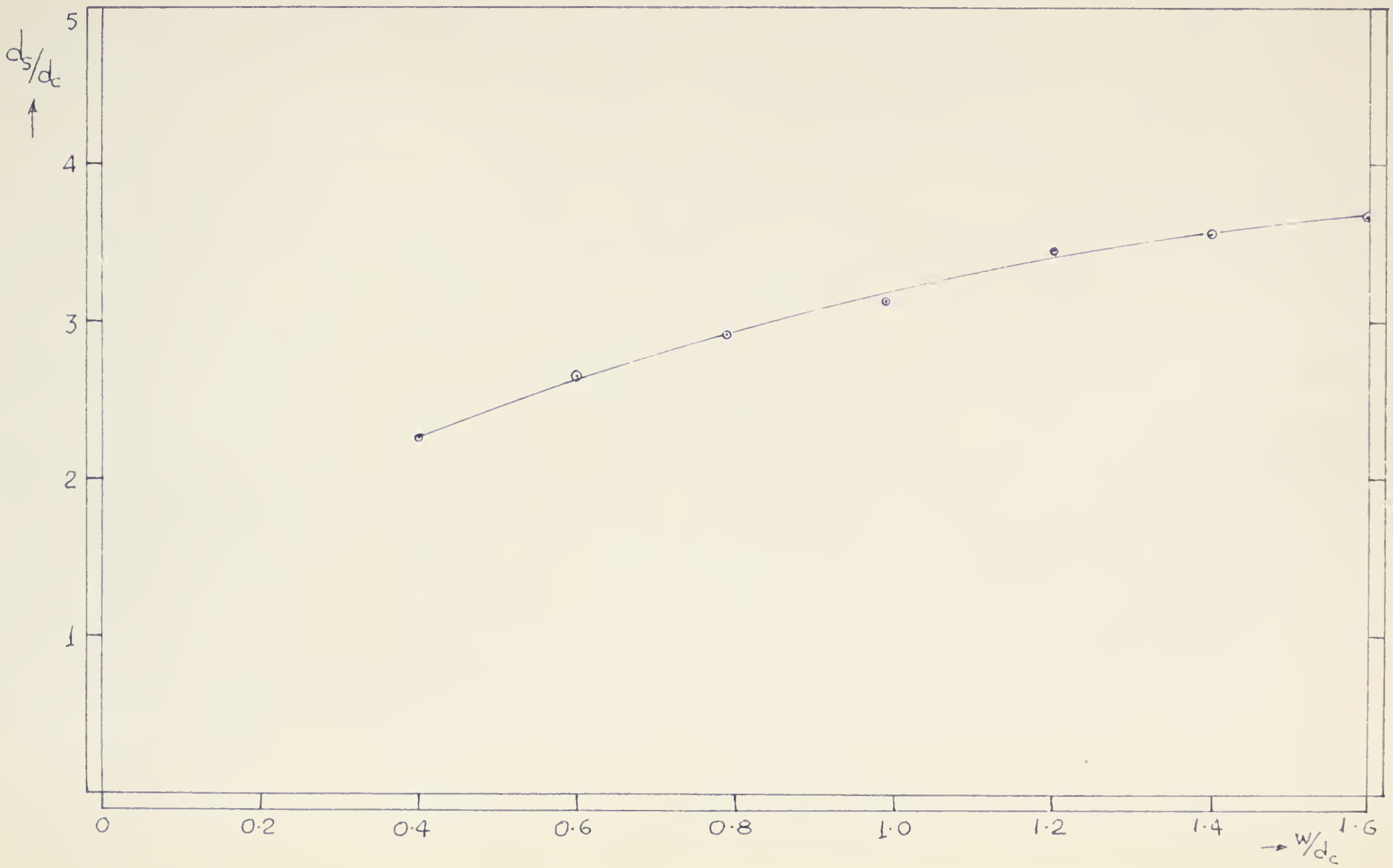


FIG. 12. EFFECT OF PIER WIDTH ON MAX. SCOURED DEPTH

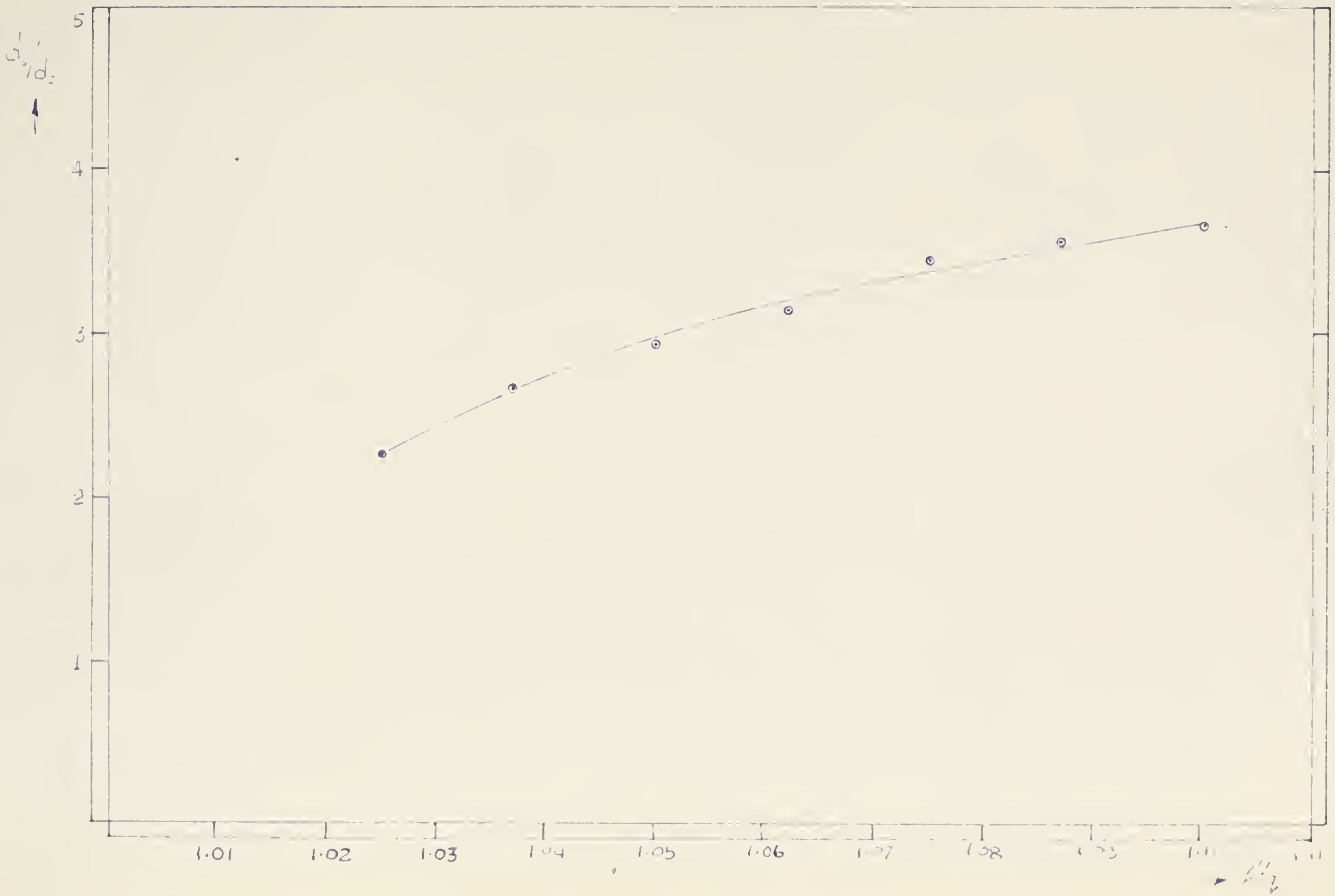


FIG 15. EFFECT OF PIER WIDTH (Plotted in terms of discharge intensity) ON MAX. SCOURED DEPTH.

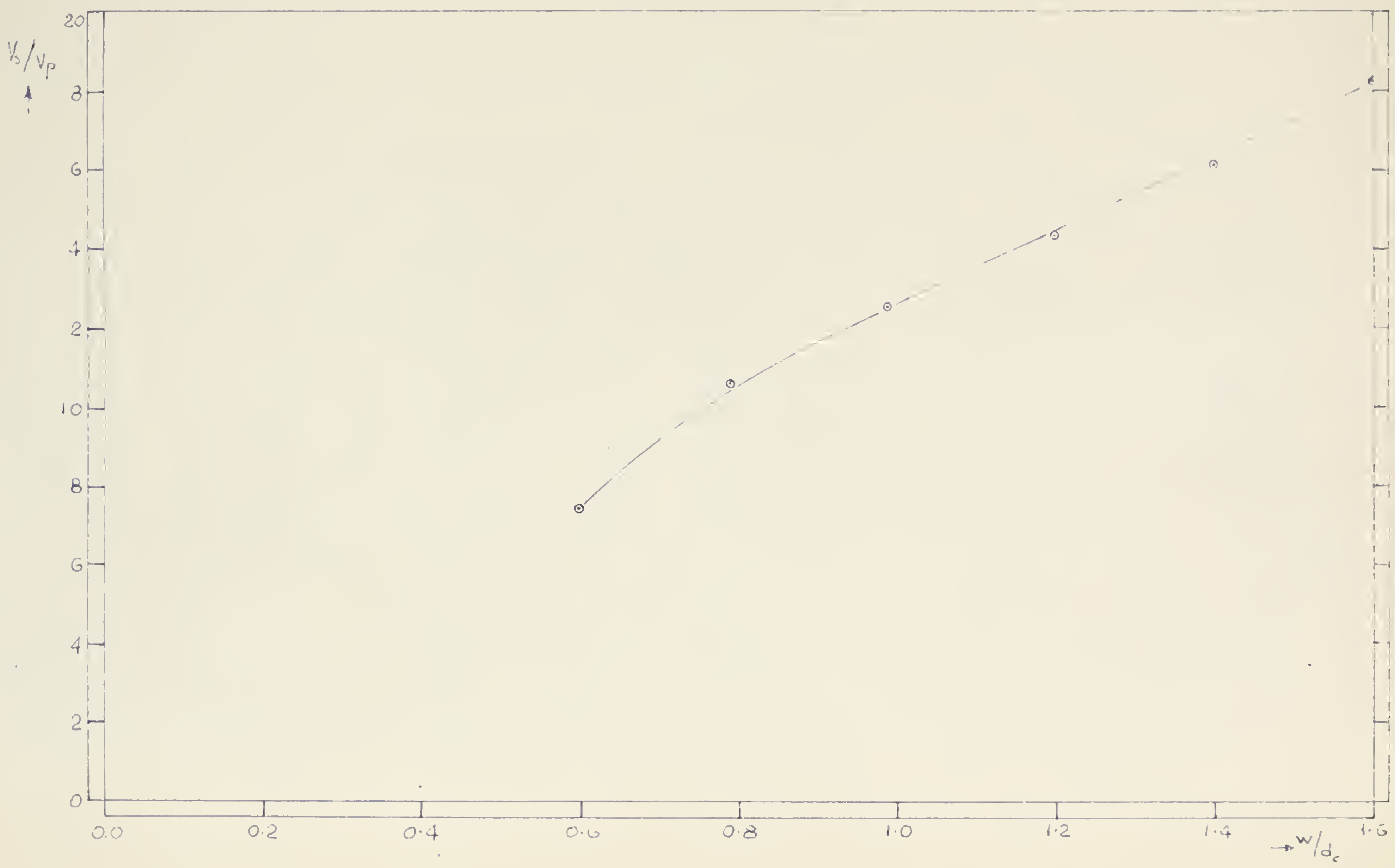


FIG 14. EFFECT OF PIER WIDTH ON SCOUR VOLUME

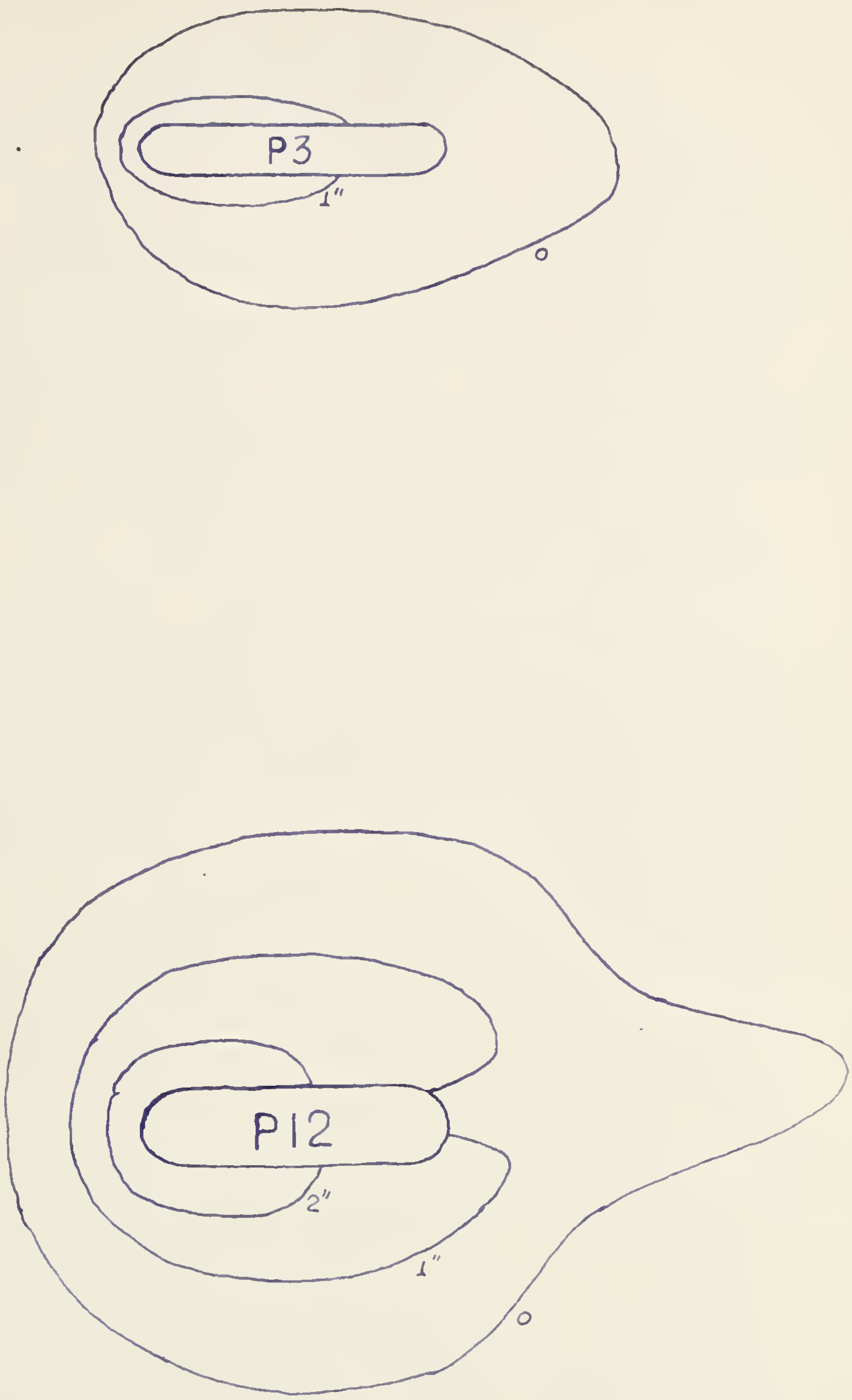


FIG 15 a & b

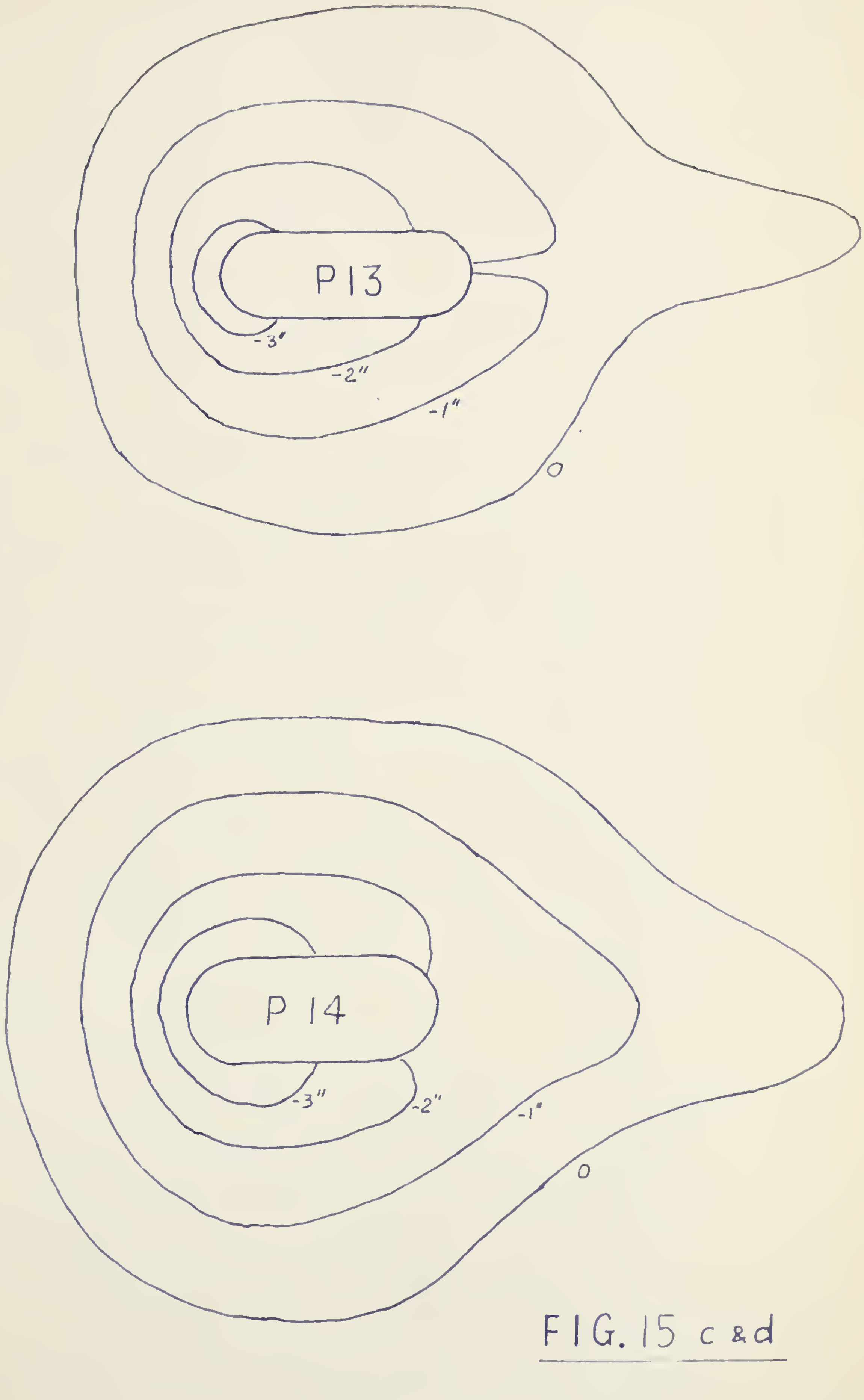


FIG. 15 c & d

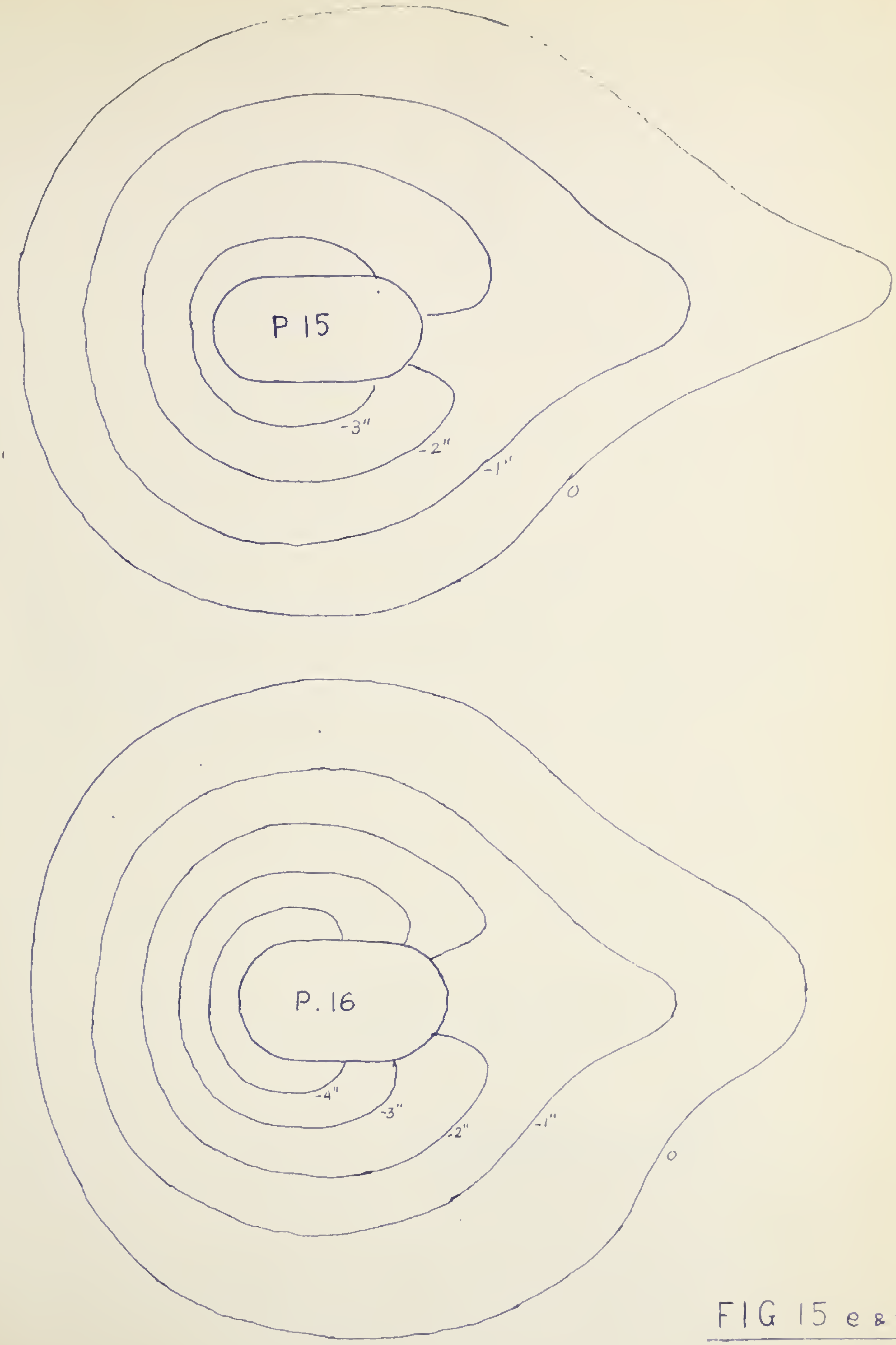


FIG 15 e & f

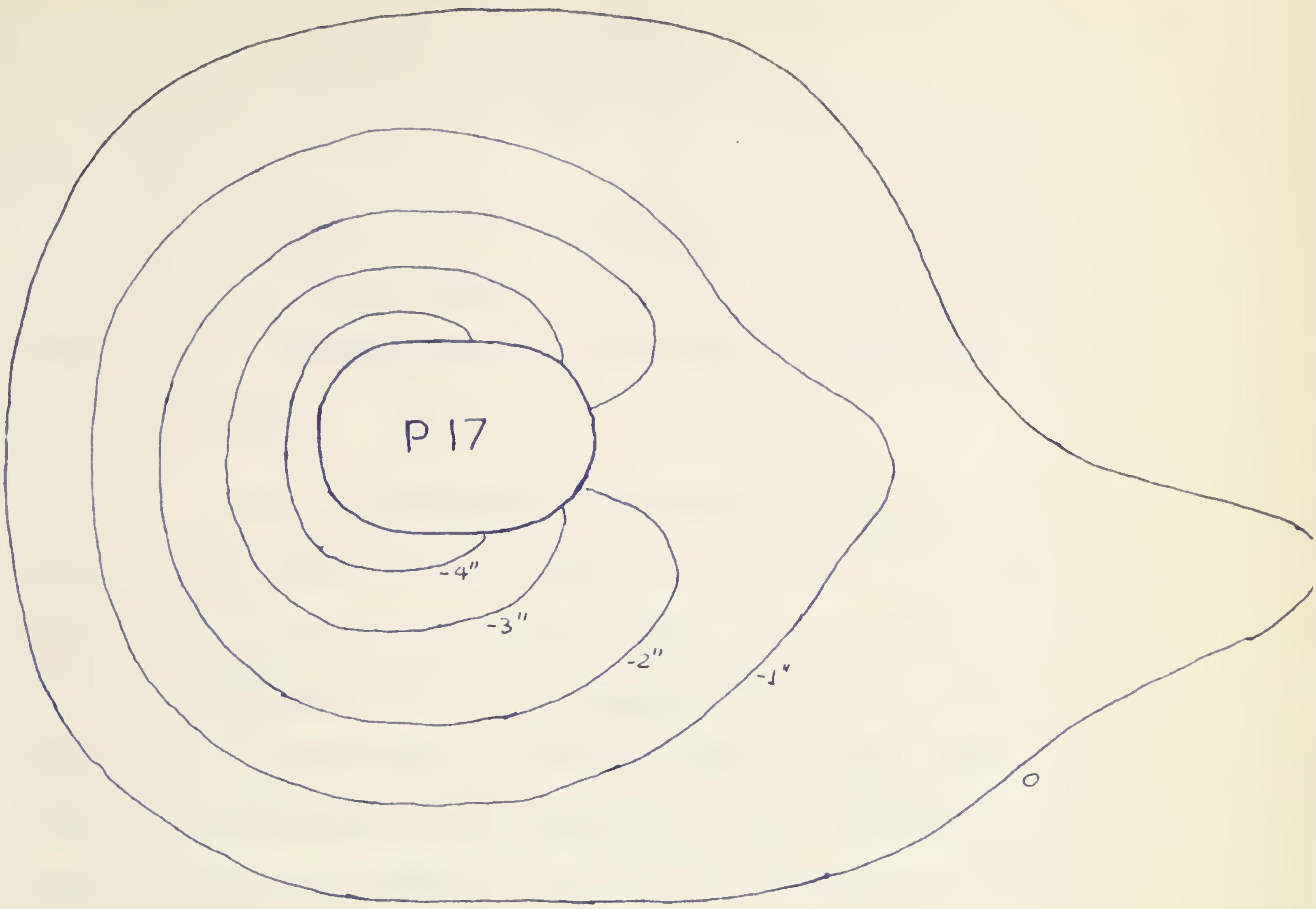


FIG 15 a

FIG 15 SCOUR PATTERNS

EXPERIMENT III

EFFECT OF SIZE OF PIER

(Experiment IV)

2.23. Actually, this is a combination of Experiments II and III. In Experiment II the effect of pier length variation was studied, and in Experiment III the effect of width variation.

Strictly speaking, in Experiment III there were two variables involved; (a) the ratio of length to width of pier and (b) the contraction of flow. In analysing the results of Experiment III it was reasoned that the scoured depth variation was due to the variation of pier width, the affect of contraction being negligible, and therefore the results showed the affect of pier width variation. (Sections 2.18 and 2.21).

2.24. In this Experiment IV there were also two variables involved; (a) the size of the pier and (b) the contraction of flow. By the same reasoning (Sections 2.18 and 2.21) the affect of contraction is debatable in this experiment as well. Therefore the results of this experiment will be considered as affected by the size of the pier. Again a proper experiment should have been run in a very wide or varying width flume as to elimate any doubt of flow contraction effect.

2.25. The equation 1.1 of section 1.23 can be written:

$$d_s = fn(1,\frac{w}{B}) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.5$$

or neglecting the affect of contraction

$$d_s = fn(l,w) \quad \ldots\ldots \quad (2.6)$$

2.26. The pier models tested in this experiment are described in Table 7 below and illustrated in Figure 16.

	Model	Shape	Overall size	Stand
(i)	P18	Rounded nose	0.5 x 1.5 inches	1
(ii)	P19	" "	1.0 x 3.0 "	2
(iii)	P20	" "	1.5 x 4.5 "	3
(iv)	P13	" "	2.0 x 6.0 "	4
(v)	P21	" "	2.5 x 7.5 "	5
(vi)	P22	" "	3.0 x 9.0 "	6
(vii)	P23	" "	3.5 x 10.5 "	7

Table 7. Model piers of Experiment IV

2.27. The experiment procedure followed in this experiment was the same as in the previous Experiments I, II and III (sec. 2.5).

2.28. The results of the test are shown and analysed in Table 8 which should be read together with Table 7 and Figures 16, 17, 18 and 19a, b, c, d, e, f, g. In Figure 17 scoured depth is plotted non-dimensionally against size of pier, that is d_s/d_c versus l/d_c. The length of pier l is a measure of the size of the pier as the ratio length to

width was constant in all pier models and equal to 3. In Figure 18 volume of scour is plotted against size of pier, that is V_s/V_p versus $1/d_c$. Finally in Figures 19a, b, c, d, e, f, and g the scour surface contours are drawn.

Model	Overall size in.	a sq.in.	L in.	d'_s in.	d in.	d_s in.	V_s c.in.	d_c in.	$\frac{d_s}{d_c}$	$\frac{1}{d_c}$	V_s c.in.	$\frac{V_s}{V_p}$
P18	0.5 x 1.5	0.70	3.57	0.80	4.20	5.00	1.5	2.51	1.99	0.595	1.5	2.14
P19	1.0 x 3.0	2.78	7.14	1.50	4.20	5.70	17.5	2.51	2.27	1.195	17.5	6.30
P20	1.5 x 4.5	5.76	10.71	2.30	4.20	6.50	49.0	2.51	2.59	1.790	49.0	8.51
P13	2.0 x 6.0	11.14	14.28	3.10	4.20	7.30	133.0	2.51	2.91	2.390	133.0	11.95
P21	2.5 x 7.5	17.40	17.85	3.50	4.20	7.70	225.0	2.51	3.07	2.990	225.0	12.93
P22	3.0 x 9.0	25.06	21.42	4.10	4.20	8.30	320.0	2.51	3.31	3.590	320.0	12.78
P23	3.5 x 10.5	34.11	24.98	4.20	4.20	8.40	350.0	2.51	3.34	4.180	350.0	10.28
1	2	3	4	5	6	7	8	9	10	11	12	13

N.B. $\frac{1}{w} = 3$ for all models. Therefore 1 is a measure of size.

Table 8. Results of Experiment IV

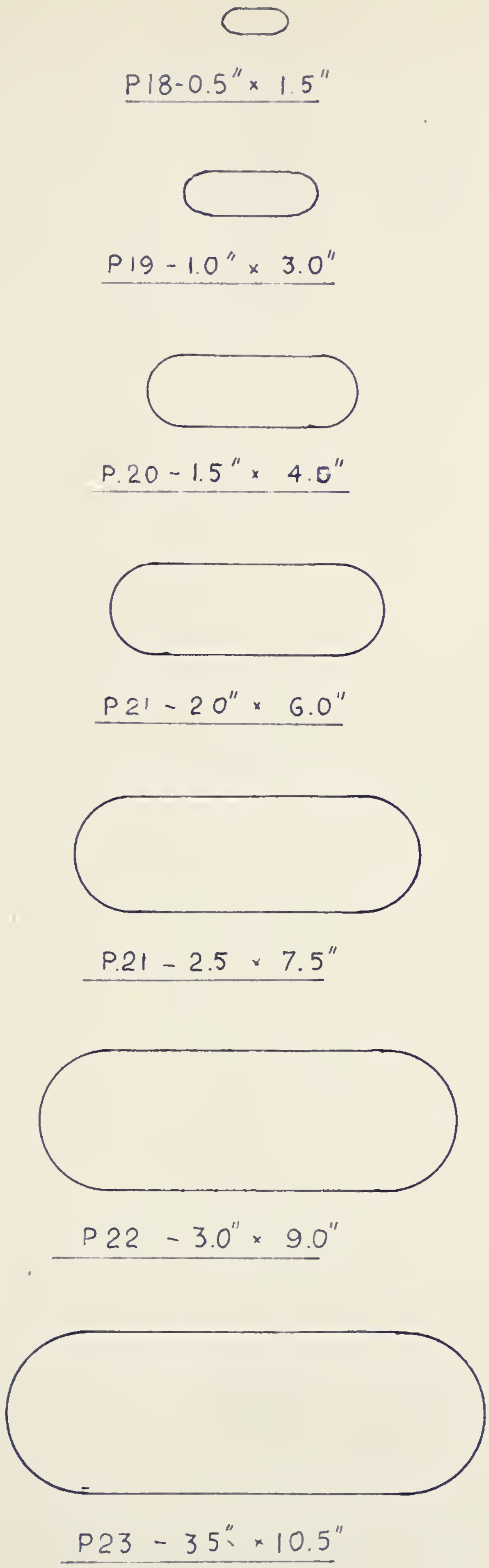


FIG 16. MODEL PIERS OF EXPERIMENT IV

SCALE: 1/8" = 1"

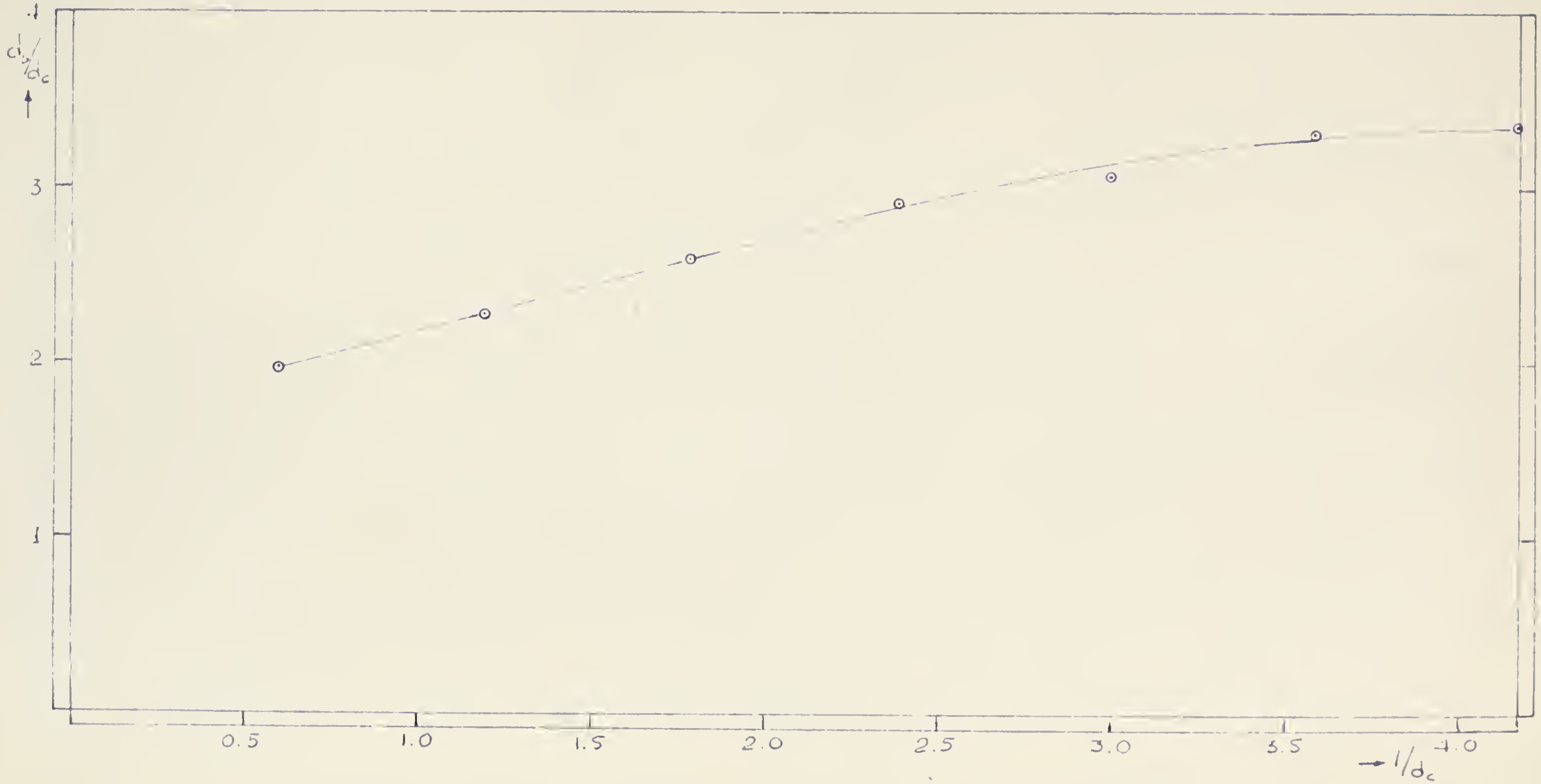


FIG 17. EFFECT OF PIER SIZE ON MAX. SCOURED DEPTH

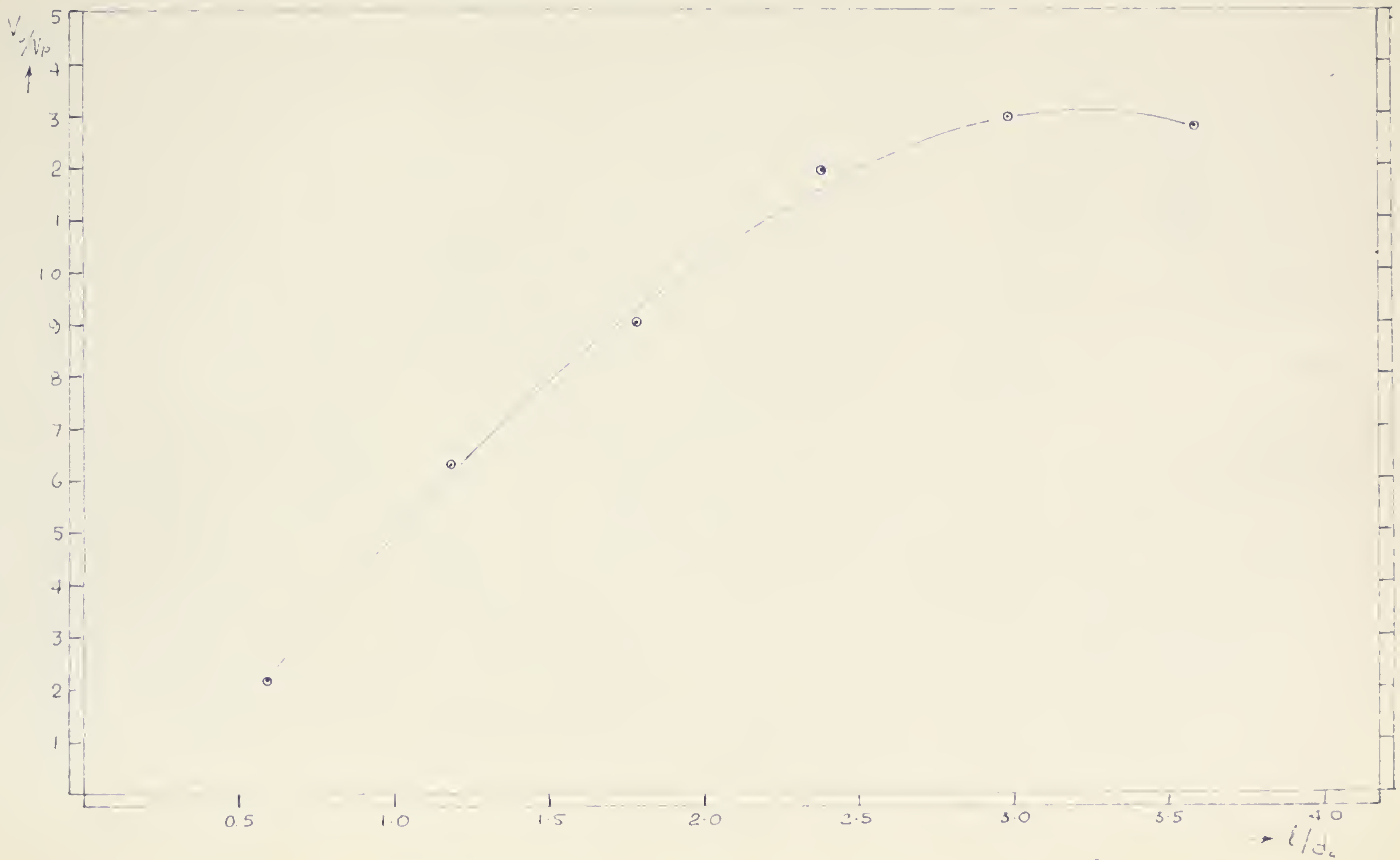


FIG 18. EFFECT OF PIER SIZE ON SCOUR VOLUME

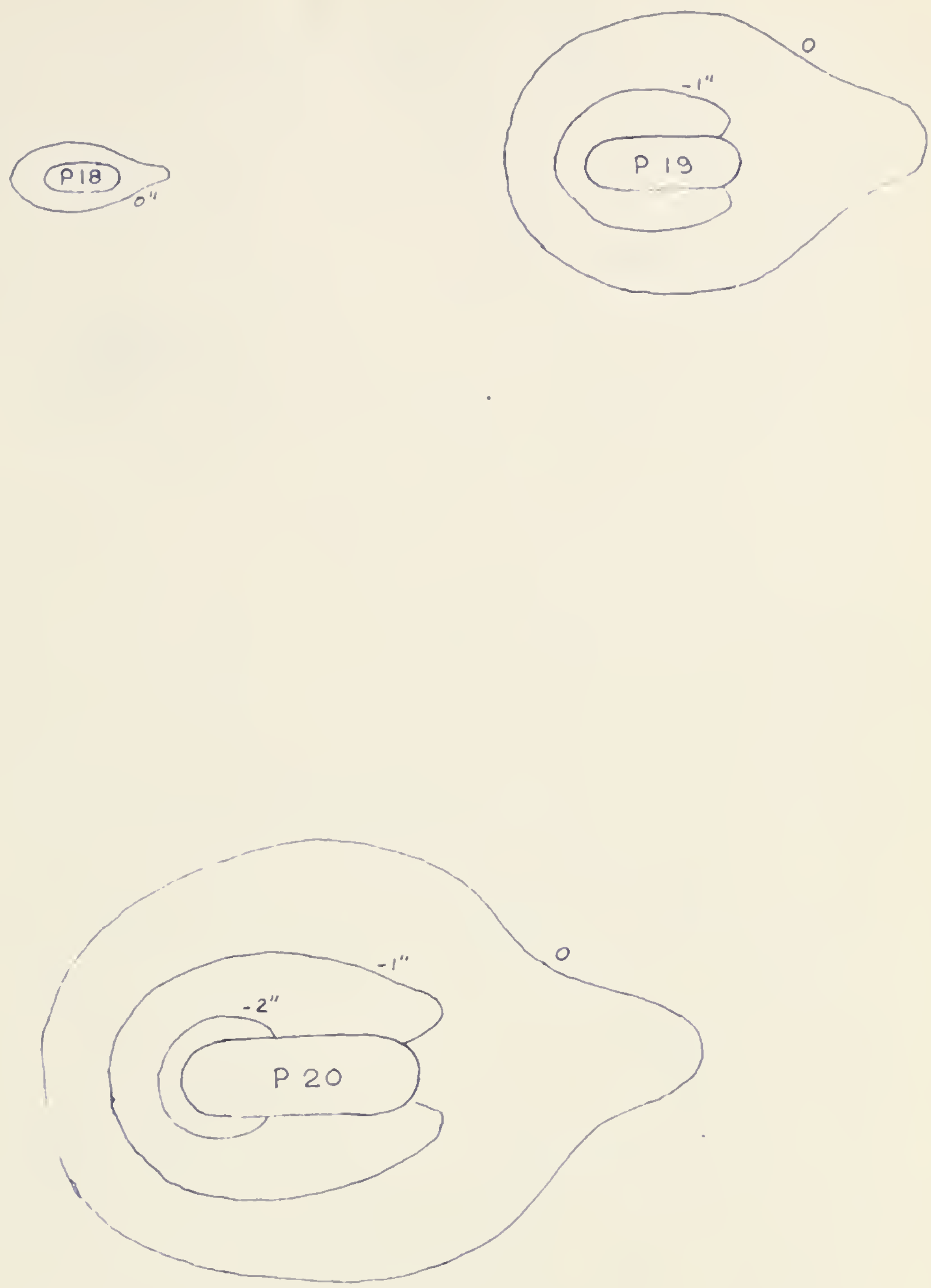


FIG 19. a, b & c.

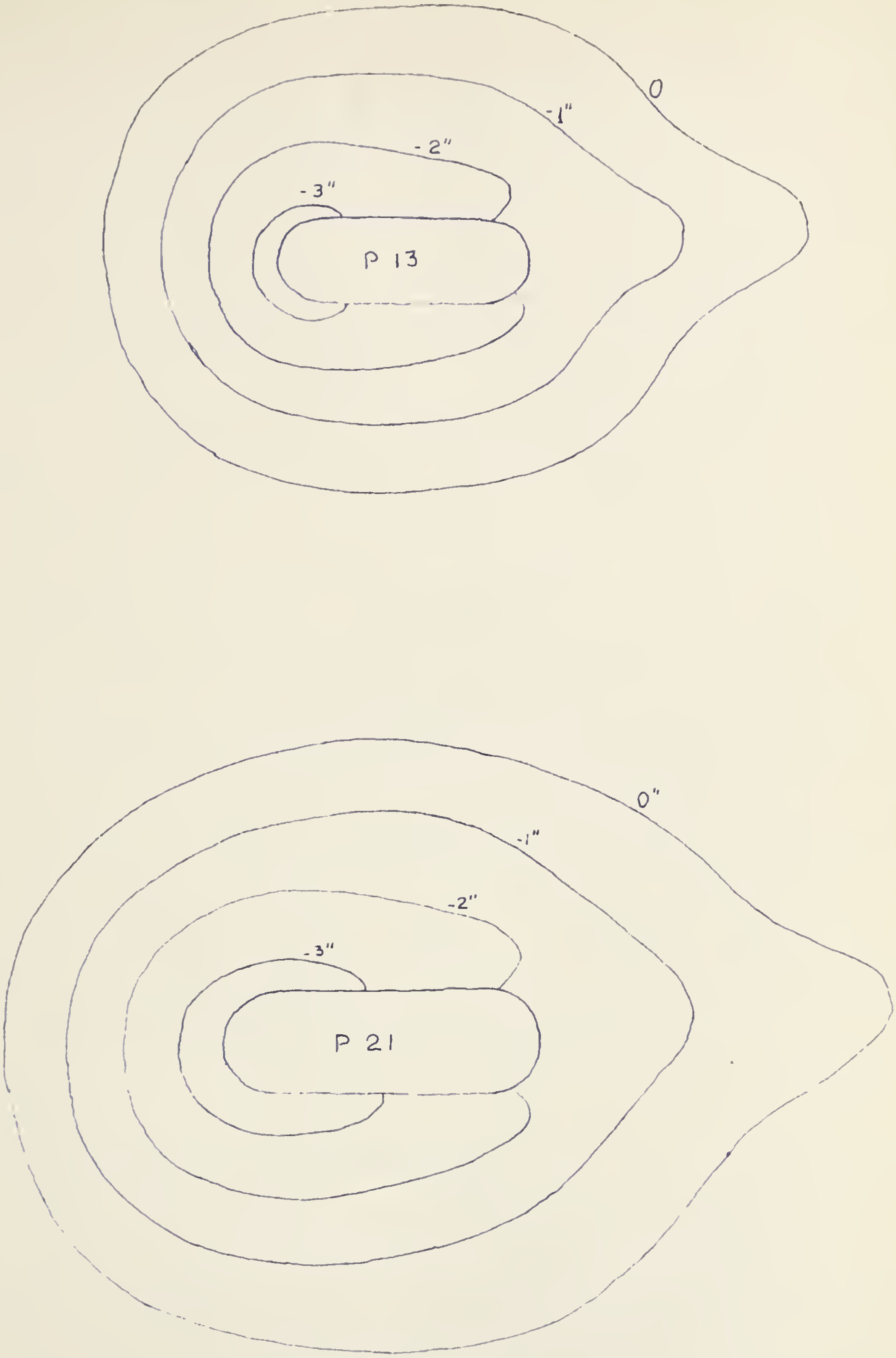
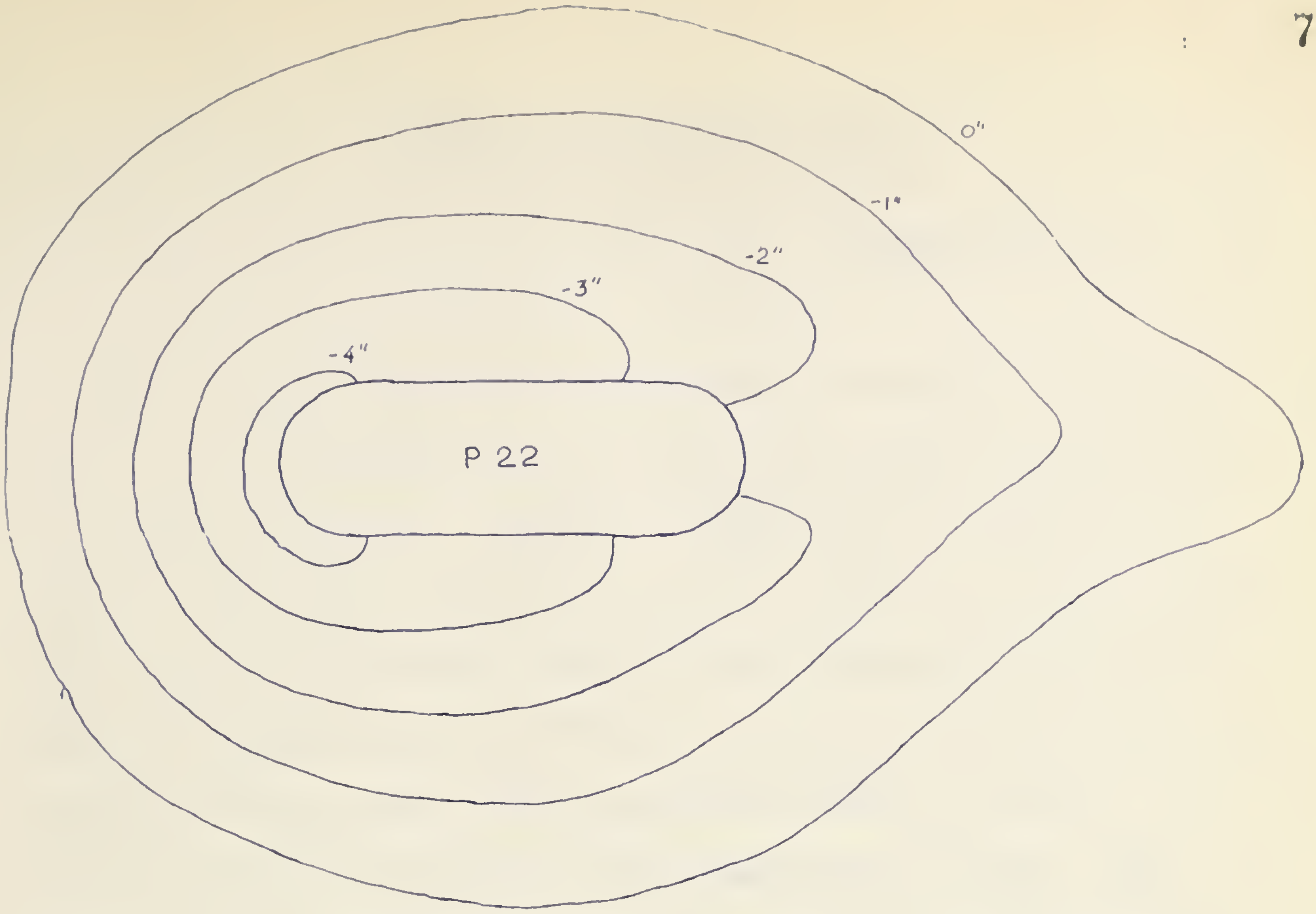


FIG. 19 d & e.



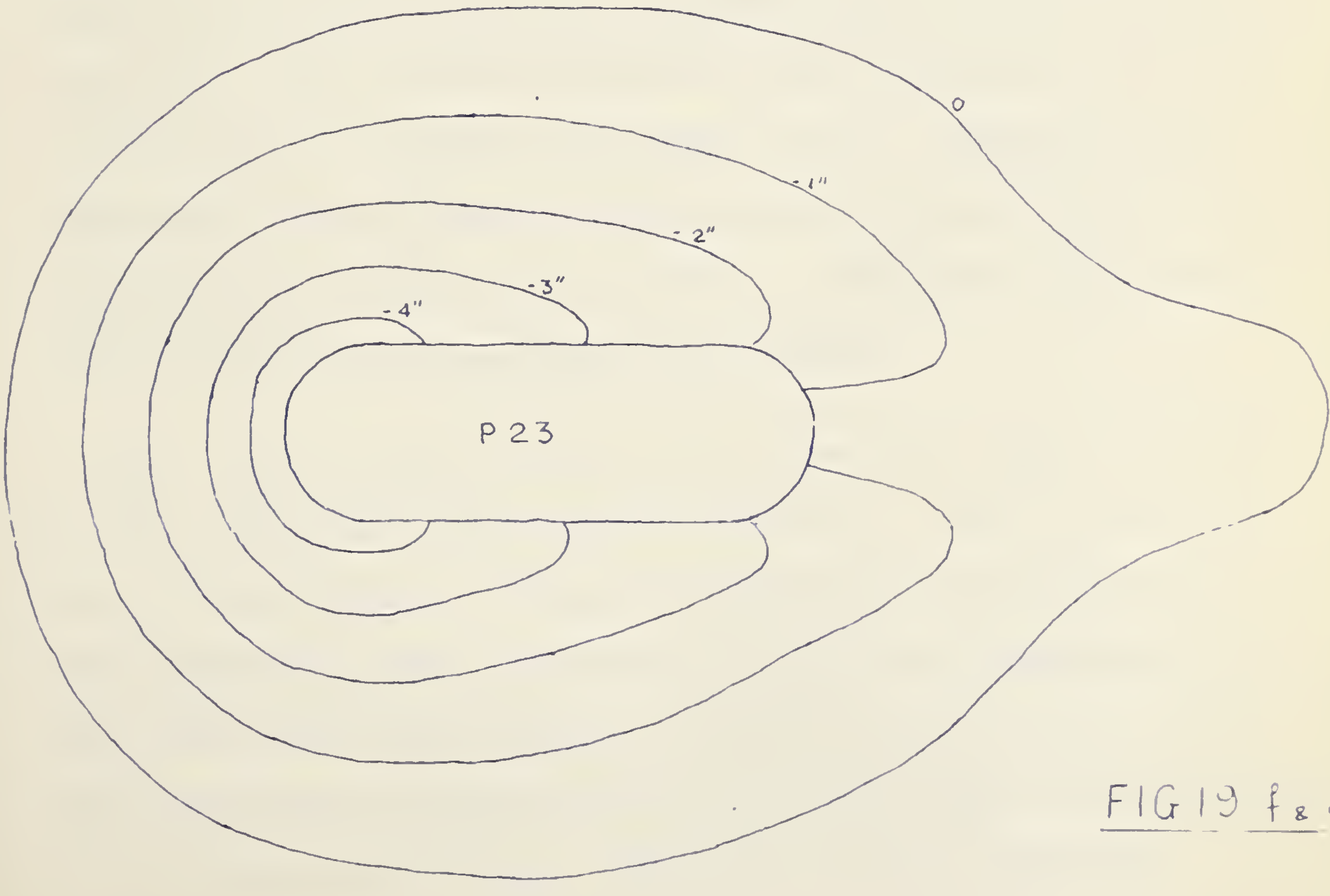


FIG 19 f & g

FIG. 19 SCOUR PATTERNS.

EXPERIMENT IV

EFFECT OF ANGLE OF ATTACK

(Experiment V)

2.29. Another factor contributing to the scour development is the alignment of the flow to the pier or, the same, the angle of attack. Seldom a bridge will be built at an angle to the mean direction of flow. Even in the case where the bridge is not built normal to the stream the piers are usually aligned with the flow. The need to study this factor arises from the fact that the stream changes course. The mean direction of flow can even change with stage. The phenomenon of meandering is well known to river engineers, hydrologists, geologists and people living by the riverside. Therefore, the stream is bound to attack the piers at a different than zero angle to the flow except, possibly, in the case where guide banks have been built and are expected to withstand the time and the floods, or where the bridge is at the head of a river emerging from a lake.

2.30. For a proper study of the effect of angle of attack a very broad flume should have been used. This was not the case, as the standard flume, as described in Section 1.33, was employed in this experiment. Still, the contraction of flow was of the same order as in Experiments III and IV, if one calculates contraction using projected width of pier; i.e. w'/B, which is debatable. The same argument regarding

contraction, as in Sections 1.48, 2.18 and 2.21 can be applied. Furthermore, the discharge intensity is bound to be different on each side of the contracted section as the pier directs flow towards one side.

After the above, the equation 1.1 of Section 1.23 can be written:

$$d_s = fn\ (\theta) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.7)$$

2.31. Seven pier models were employed in the experiment, all of them of the same overall size and shape; i.e. 1.0 x 6.0 inches size and rounded nose shape (P3, Fig. 8). The pier on Stand No. 1 was aligned at zero angle to the flow, the pier on Stand No. 2 at 7.5° to the right, No. 3 at 15° to the left, No. 4 at 22.5° to the right, No. 5 at 30° to the left, No. 6 at 37.5° to the right, No. 7 at 45° to the left. This arrangement was adopted so that not all the piers would direct the flow towards one side. It proved to be effective. Except for the different than zero angles of attack, the experiment was conducted in the same manner as the previous ones. (Section 2.5).

2.32. The results of this experiment appear, and are discussed, on Table 9, and Figures 20, and 21, 22, 23, 24 and 25. In Figure 20, scoured depth, divided by the same of the model pier aligned with the flow, is plotted against angle of attack.

In Figure 21, scoured volume, divided by scoured volume of the pier alignment with the flow, is plotted non-dimensionally against angle of attack.

Therefore, the coefficients Ks and Kv, are given, by which the scoured depth and scour volume, respectively should be multiplied to include the effect of anticipated angle of attack.

These coefficients have been obtained from experimental models of 1:6 pier width to length ratio, and should not be expected to apply to other ratios. In order to include other pier width to length ratios a good approximation should be to allow for increasing or decreasing projected width w', according to Figs. 12, 14 or 17 and 18.

2.33. It is worth mentioning that the place where the maximum scoured depth occurs is near the downstream nose and on the exposed side, while in piers aligned with flow it occurs right on the upstream nose. This is shown in Fig. 25a, b, c, d, e, f, g, which show the contours of the scour surfaces, produced around the model piers of this experiment.

2.34. Sir C. C. Inglis (Ref. 2) uses projected pier width in place of width in order to include the effect of angle of attack in his formula for prediction of scour. The meaning of projected width is illustrated in Fig. 22. In the case investigated, this practice could lead to an error in excess

of 30%, as illustrated in Fig. 23 where the scour data of this Experiment V are plotted against "projected width" (w') and the scour data of Experiment III (Effect of width variation) are plotted against width w, (w and w' measured on the same scale). Note that for the same pier aligned at zero angle to the flow results from both Experiments III and V could be said to coincide.

2.35. Furthermore, the results of this experiment were compared with similar experiments of Laursen and Toch's publication (Ref. 4), as shown in Fig. 24. A major discrepancy exists. Each of the curves turns its concave toward the other. The fact that Laursen's pier was rectangular, while in this experiment the model was of rounded ends, does not explain the discrepancy. Possible explanations are that Laursen and Toch's sand was of different size and constitution and they did not allow time for the scour to fully develop.

Model	θ degrees	Over-all size	a sq.in.	L in.	w' in.	d'_s in.	d in.	d_s in.	V_s	$\frac{d_s}{d_{s0}}$ *	$\frac{V_s}{V_{s0}}$ *	$\frac{d'_s}{d_{s0}}$
P3	0.0°	1 x 6	5.78	13.14	1.000	1.38	4.20	5.58	18	1.000	1.00	1.00
P3	7.5°	1 x 6	5.78	13.14	1.653	1.63	4.20	5.83	43	1.045	2.39	1.18
P3	15.0°	1 x 6	5.78	13.14	2.294	1.88	4.20	6.08	83	1.091	4.60	1.36
P3	22.5°	1 x 6	5.78	13.14	2.913	2.37	4.20	6.57	105	1.179	4.82	1.72
P3	30.0°	1 x 6	5.78	13.14	3.500	3.25	4.20	7.45	205	1.335	11.40	2.36
P3	37.5°	1 x 6	5.78	13.14	4.044	4.15	4.20	8.35	420	1.495	23.35	3.01
P3	45.0°	1 x 6	5.78	13.14	4.536	5.20	4.20	9.40	675	1.685	37.50	3.77
1	2	3	4	5	6	7	8	9	10	11	12	13

* The subscript 0 means that the quality refers to pier of zero alignment.

Table 9. Results of Experiment V

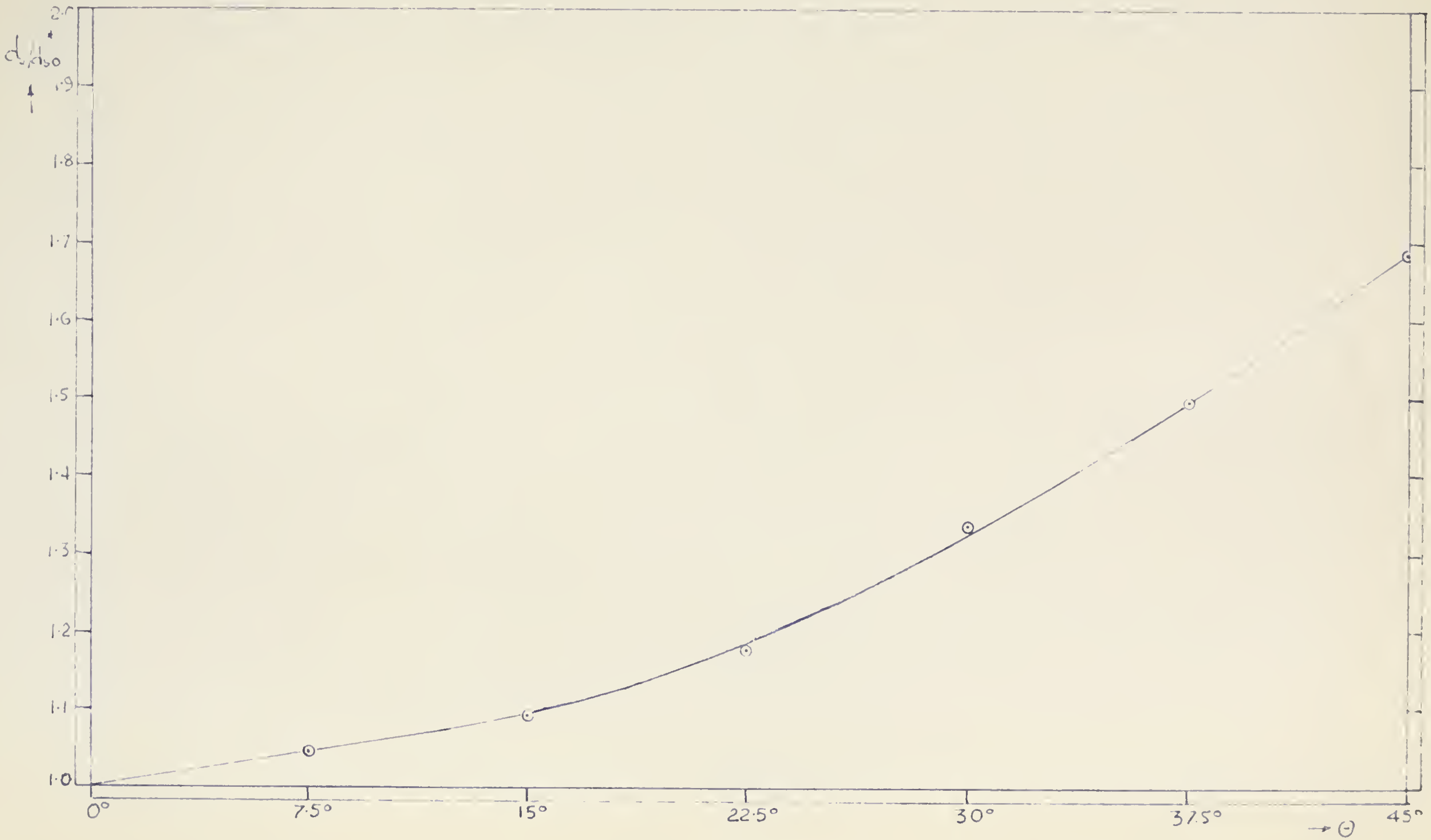


FIG. 20. EFFECT OF ANGLE OF ATTACK ON MAX SCOURED DEPTH

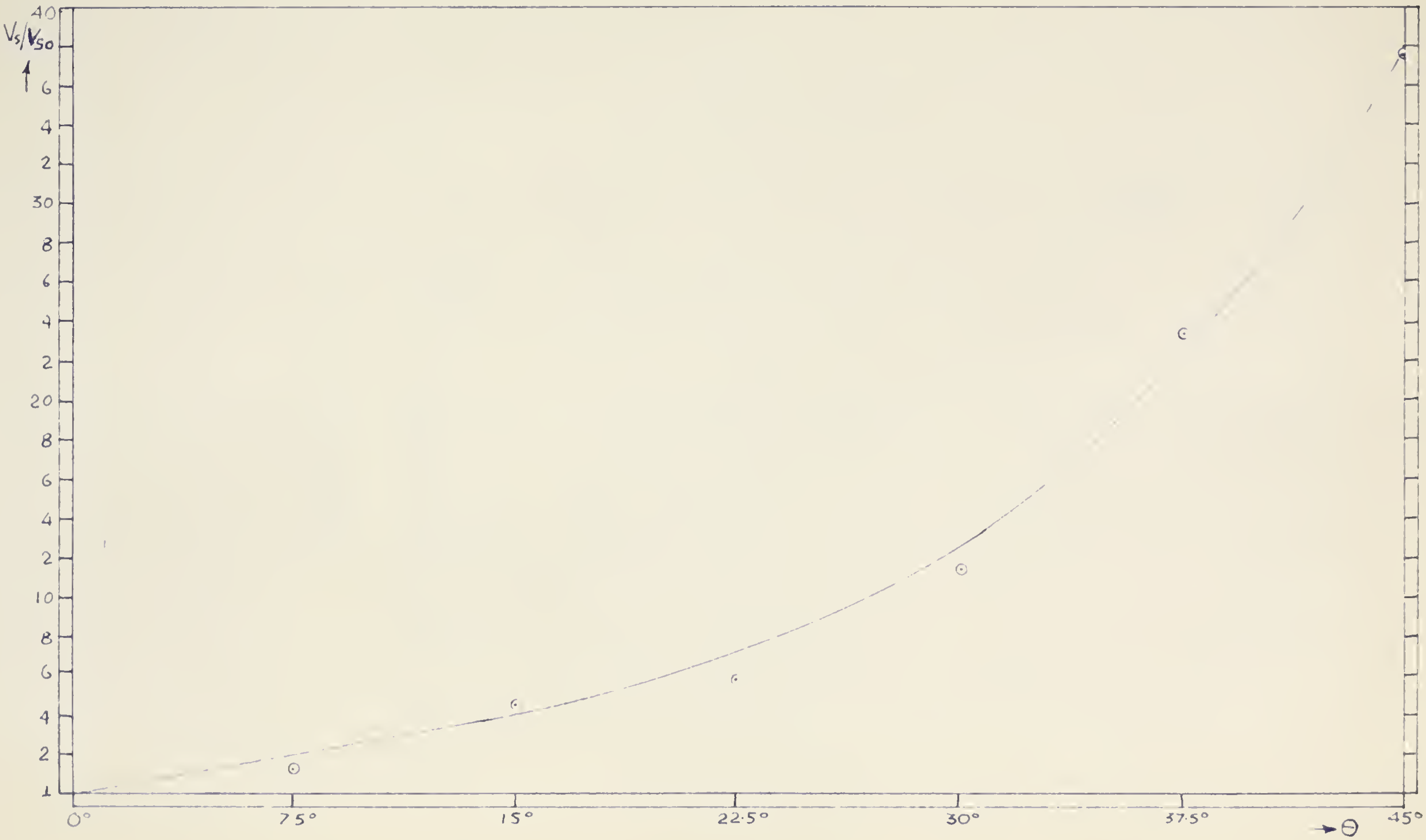


FIG. 21 EFFECT OF ANGLE OF ATTACK ON SCOUR VOLUME

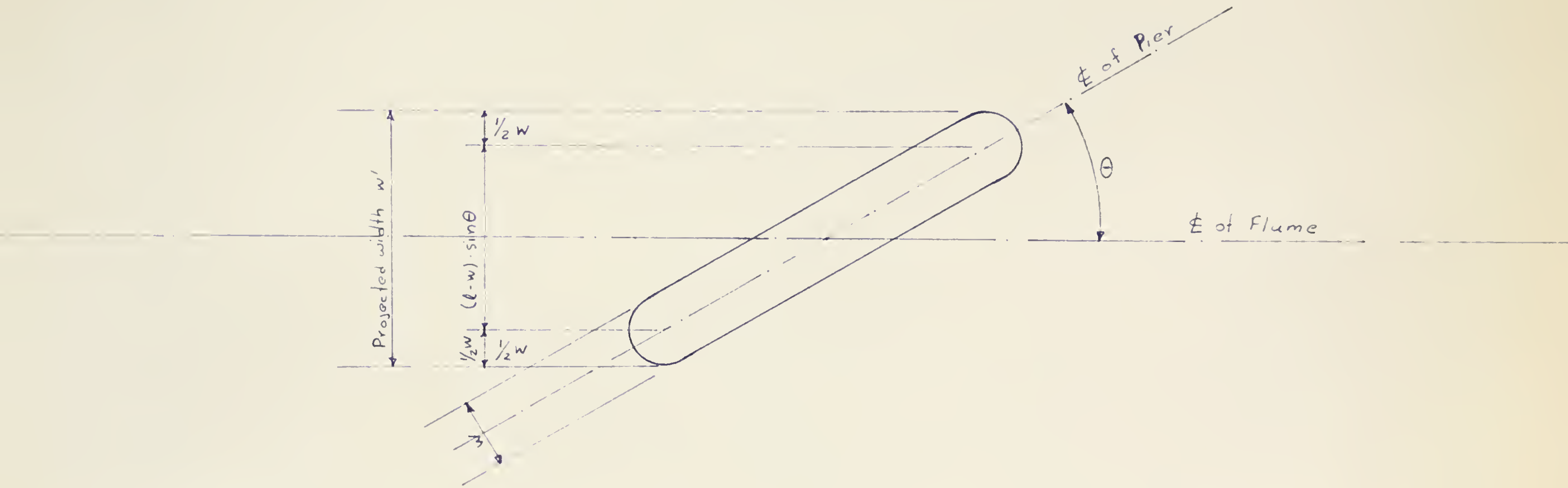


FIG. 22. EXPLANATION OF THE TERM "PROJECTED WIDTH (W')"

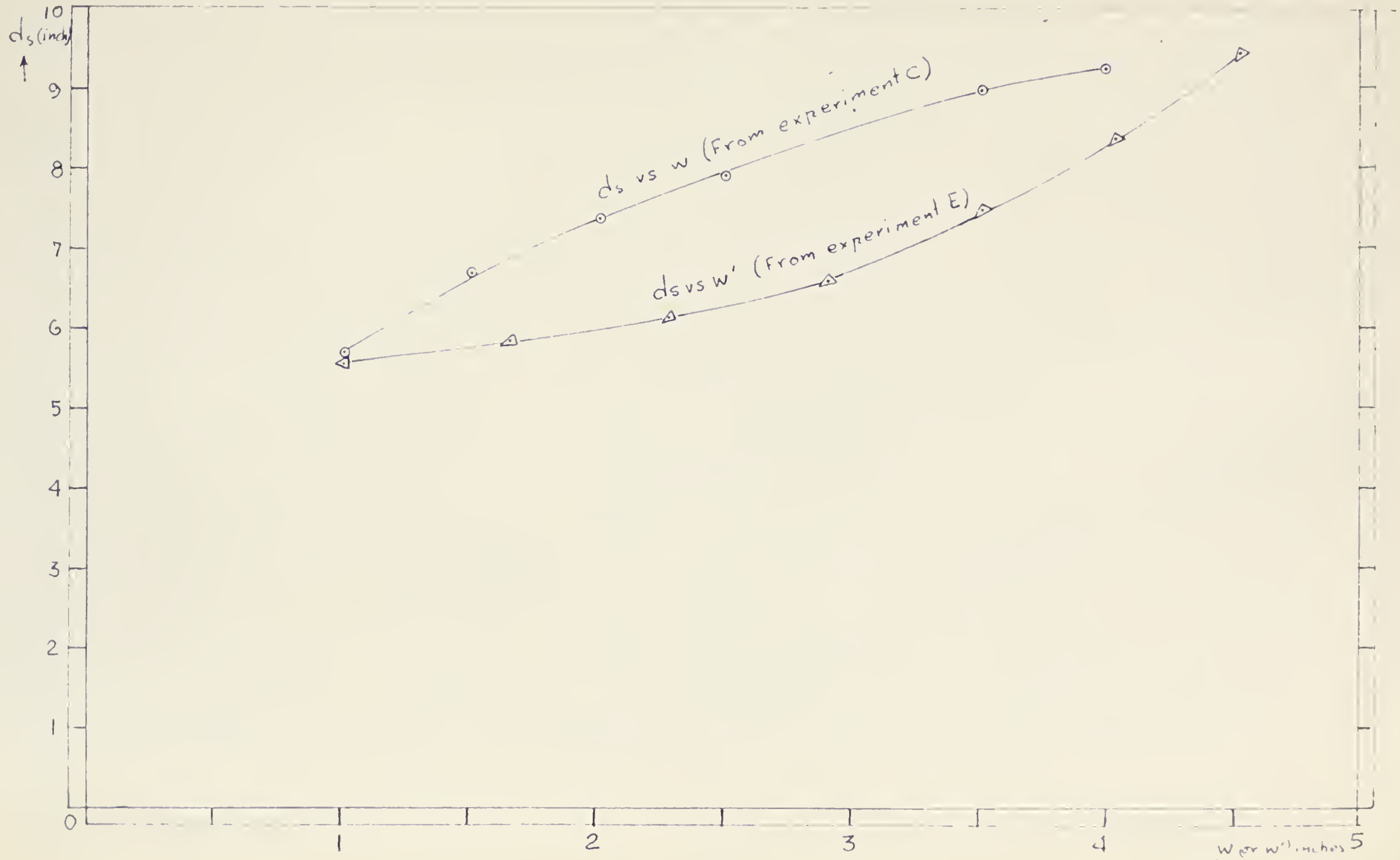


FIG. 23 MAX SCOURED DEPTH vs WIDTH OF PIER AND PROJECTED WIDTH OF PIER

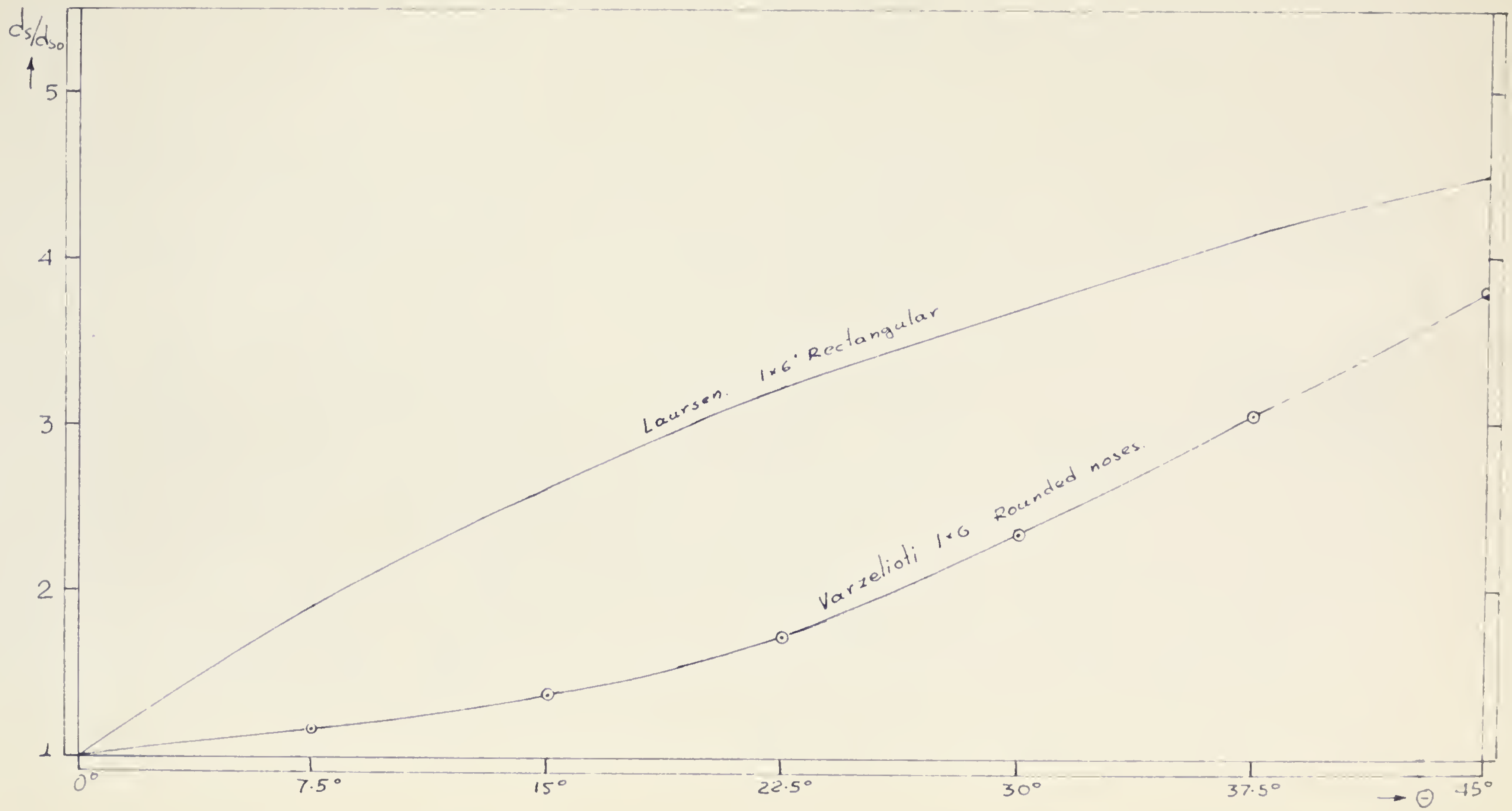
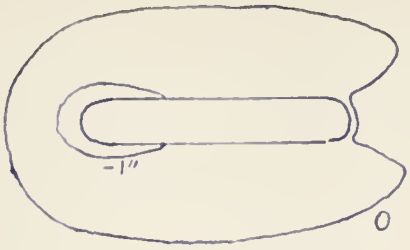
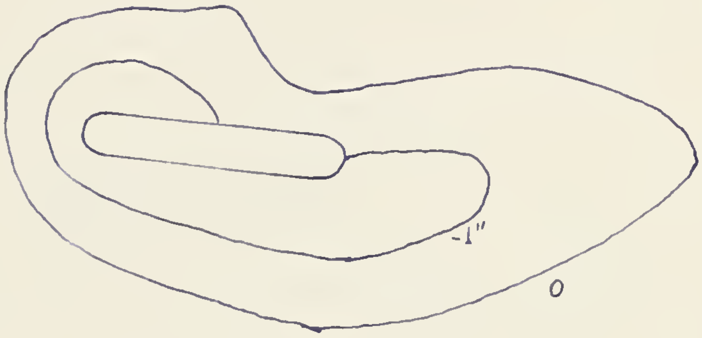


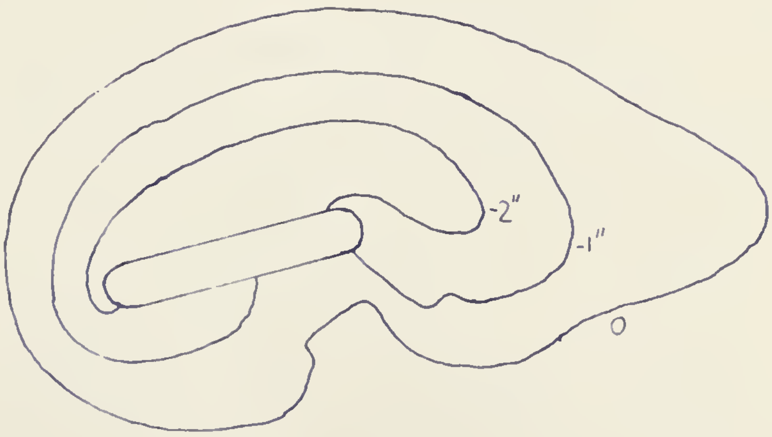
FIG. 24 COMPARISON WITH LAURSEN'S DATA ON ANGLE OF ATTACK EFFECT



P3 at 0°

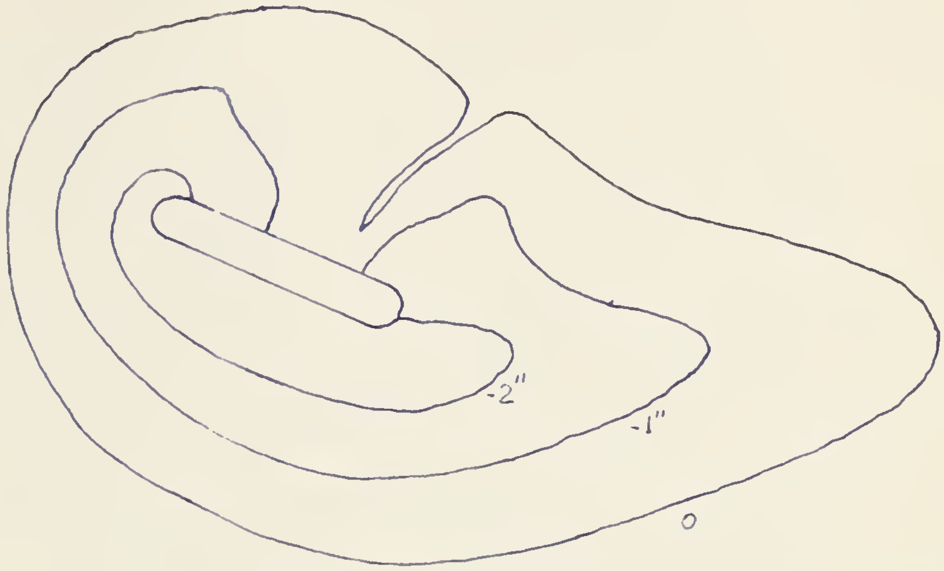


P3 at 7.5°

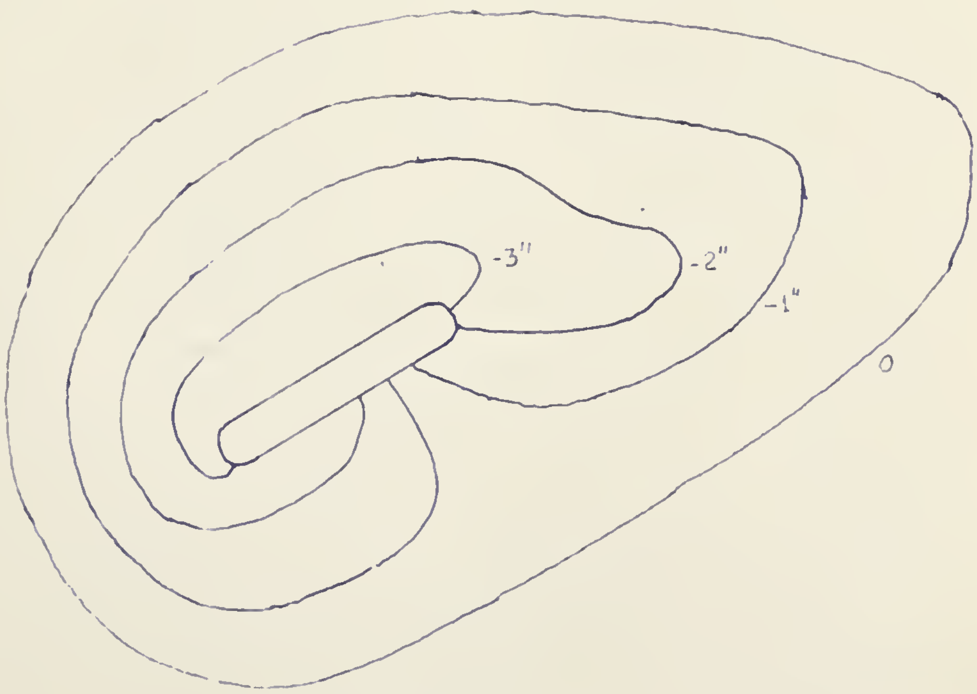


P3 at 15°

FIG 25 a, b & c.

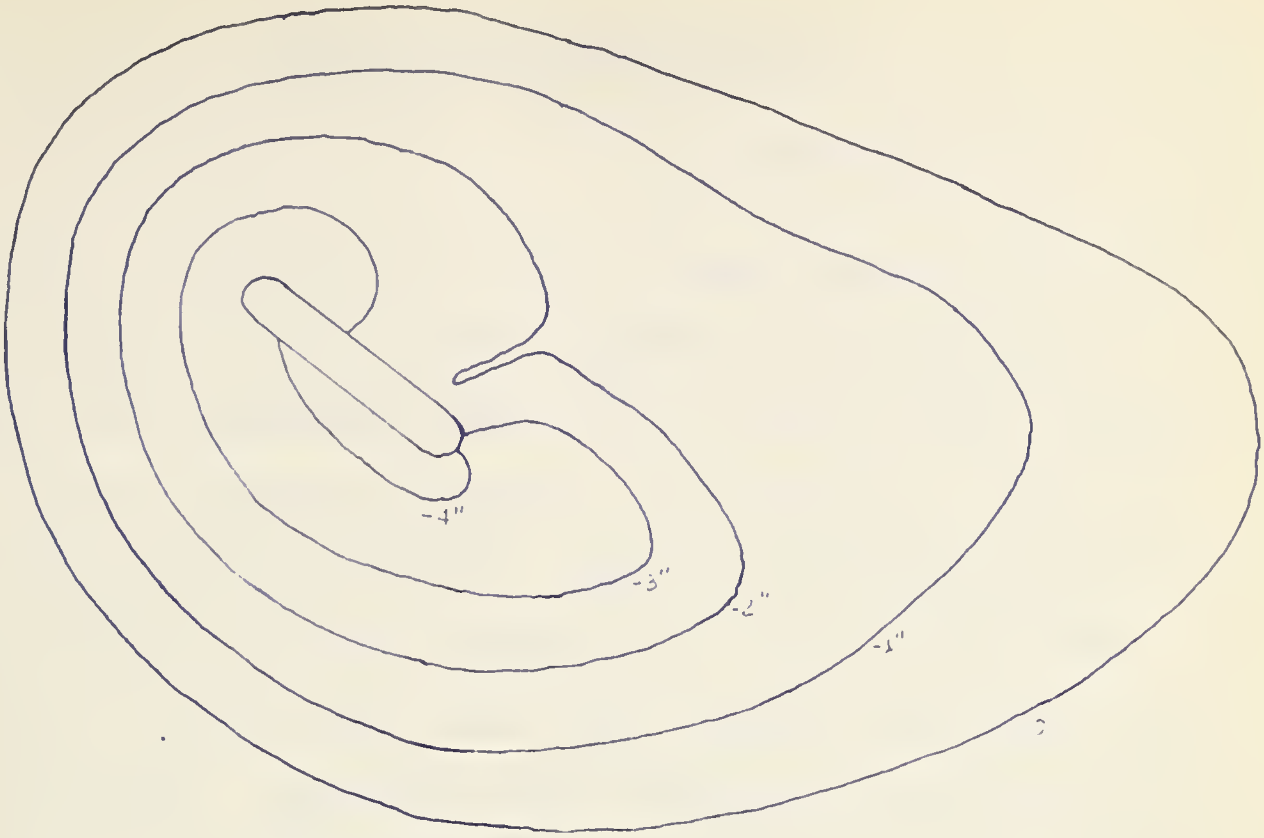


P3 at 22.5°



P3 at 30°

FIG 25 d & e



P3 at 37.5'

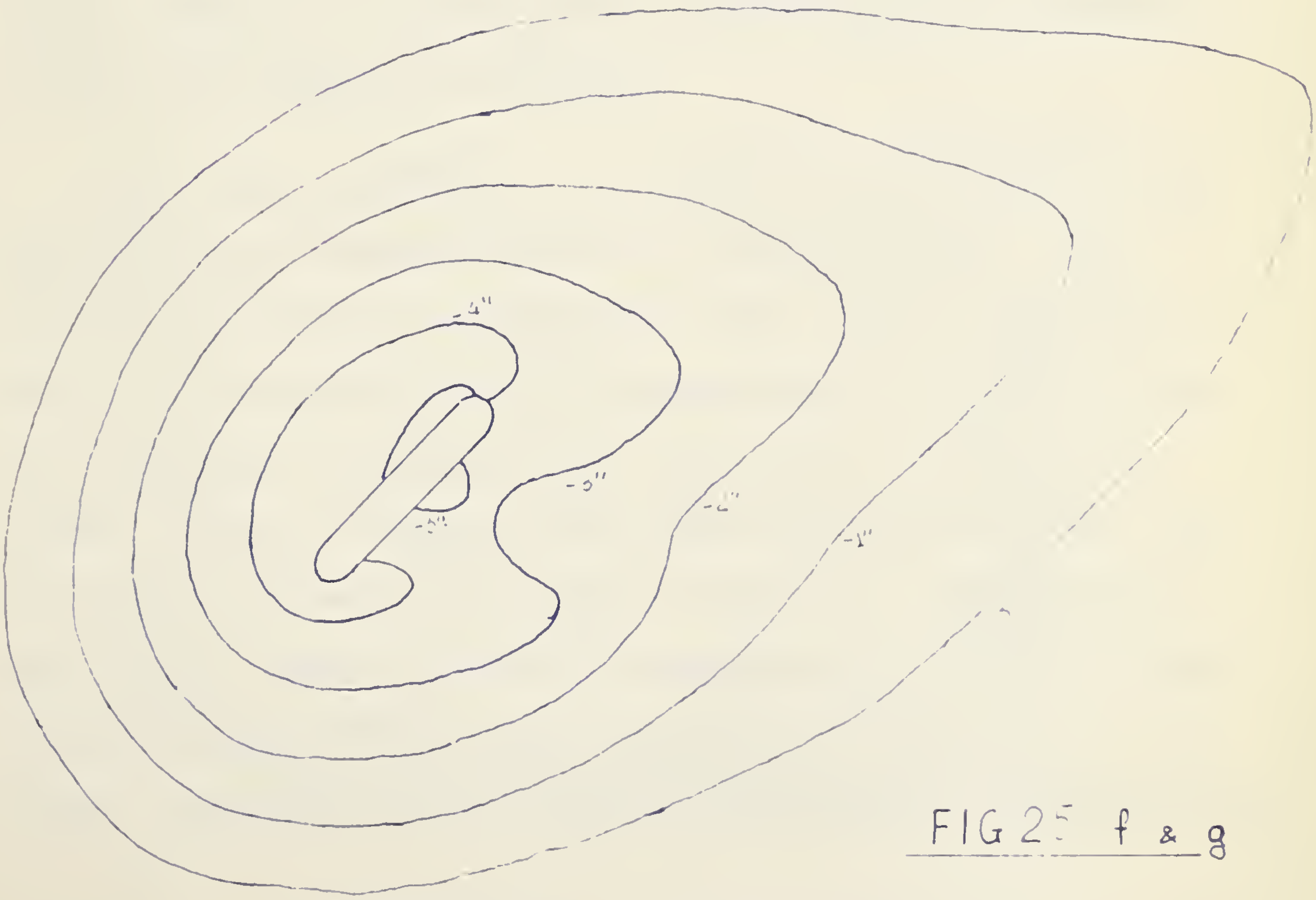


FIG 25 f & g

P3 at 45°

FIG 25. SCOUR PATTERNS

EXPERIMENT V

EFFECT OF FLOW CONTRACTION

(Experiment VI)

2.36. In previous sections of this thesis (1.48, 2.18 2.21, 2.23, 2.24 and 2.30), arguments were presented to support the assumption that the flow contraction, caused by different width pier models, had negligible effect on the relative scour studies. Indeed all previous experiments should have been conducted in a very broad flume instead of the 44 inch wide flume employed throughout this investigation. Therefore flow contraction existed, on the sides of the model piers, contributing to the scour development, although it was neglected in the analyses on grounds of the aforementioned arguments.

2.37. This experiment is devoted to showing whether the neglected flow contraction factor was negligible. This experiment differs from Experiment VII, which is presented next, as the water surface slope was not kept constant for all discharge intensities as in VII; that is, backwater effect existed as imposed from flow contraction alone. The locations of contractions were the same as in the previous experiment, though contraction extended up to 22.75%. Therefore, the magnitude to which contraction contributed to the scour development of the previous experiments, and generally

the effect of contraction on scour, was studied. Equation 1.1 of Section 1.23 can be written for this experiment as:

$$d_s = fn\left(\frac{B}{b}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.8)$$

where B is the width of the unrestricted flume (B = 44 inches) and b is the restricted waterway as shown in Figure 26.

2.38. In this experiment seven pier models, all of the same shape and size, were employed; that is, model P13 (Fig.11) of 2.0 x 6.0 inches overall size and rounded nose. The flume was restricted on both sides opposite each pier by plywood restriction, something like abutments. Table 10, below, describes the setup and should be read together with Figure 26.

Model	Overall Size	Stand	Restriction (each side)
P13	2 x 6 inches	1	0.0 inches
P13	2 x 6 inches	2	0.5 inches
P13	2 x 6 inches	3	1.0 inches
P13	2 x 6 inches	4	1.5 inches
P13	2 x 6 inches	5	2.0 inches
P13	2 x 6 inches	6	3.0 inches
P13	2 x 6 inches	7	4.0 inches

Table 10 - Model piers and flow restrictions of Experiment VI

The experimental procedure followed in this experiment was the same as for the previous ones (Sec. 2.5).

2.39. The results of this experiment appear in Table 11 and Figure 27, which should be read jointly with Figure 26 and Table 10.

An explanation of the symbols used in Table 11 is considered necessary. In Column 5 the degree of contraction is presented as percentage of the unrestricted width B; q and q' are the discharge intensities in the unrestricted and restricted waterways respectively, therefore $(q' - q)\,100/q$ is the increase of discharge intensity in the restricted waterways in percent of q; d_c and d_c' are the critical depth calculated from q and q', respectively, therefore $(d_c' - d_c)100/d_c$ is the increase, in percent of d_c, of the critical depth in the contracted waterway. $q' = fn(\text{restriction})$ and $d_c' - fn(q') = fn(\text{restriction})$.

2.40. In Figure 27, scoured depth divided by critical depth of the unrestricted section is plotted against percentage of flow restriction, that is d_s/d_c versus $(B-b)100/B$. It shows that contraction of flow has no appreciable effect on scour, especially below 10% contraction. From Table 11 it is easily seen that increase of contraction from 4.55% to 9.10% affects the scoured depth by only a small quantity

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difficult to measure. Increase of contraction from 4.55% to 22.75%, will cause an increase in scoured depth by only 6.05%; this corresponds to placing a pier of 10 inches width in our 44 inch wide flume.

Scour volume was almost constant for all piers therefore it was not plotted.

2.41. After the above, it can be said with certainty that the arguments of the previous sections regarding affect of contraction stand good, and therefore the flume of 44 inch width can be considered "broad" for modeling piers not exceeding 4.5 inches in width.

Model	Overall size ins.	B in.	b in.	$\frac{B-b}{B}$ (x100)	q' cfs/f	$\frac{q'-q}{q}$ (x100)	d'_c ins.	$\frac{d'_c-d_c}{d_c}$ (x100)	d'_s ins.	d_s ins.	$\frac{d_s}{d_c}$	V_s
P13	2 x 6	44	42	4.55	0.570	4.58	2.59	3.19	3.06	7.26	2.90	135
P13	2 x 6	44	41	6.81	0.585	7.35	2.64	5.18	3.06	7.26	2.90	135
P13	2 x 6	44	40	9.10	0.600	10.10	2.68	6.77	3.12	7.32	2.92	135
P13	2 x 6	44	39	11.38	0.615	12.85	2.73	8.36	3.18	7.38	2.94	135
P13	2 x 6	44	38	13.65	0.631	15.80	2.78	10.75	3.24	7.44	2.96	135
P13	2 x 6	44	36	18.20	0.666	22.20	2.88	14.75	3.36	7.56	3.02	150
P13	2 x 6	44	34	22.75	0.705	29.35	2.99	19.50	3.50	7.70	3.07	150
1	2	3	4	5	6	7	8	9	10	11	12	13

Table 11 - Results of Experiment VI

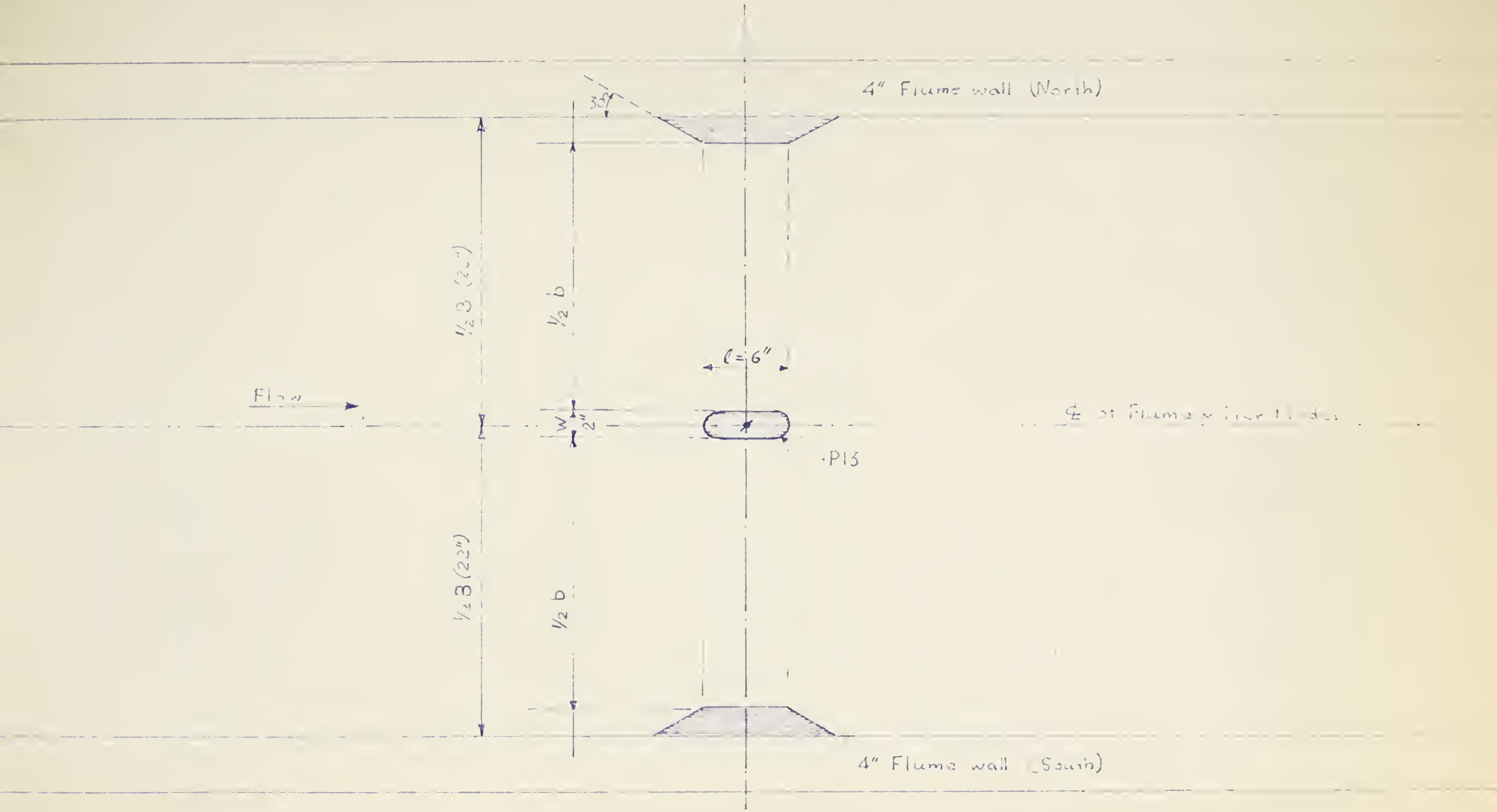

FIG 26. ARRANGEMENT FOR STUDY OF CONTRACTION EFFECT

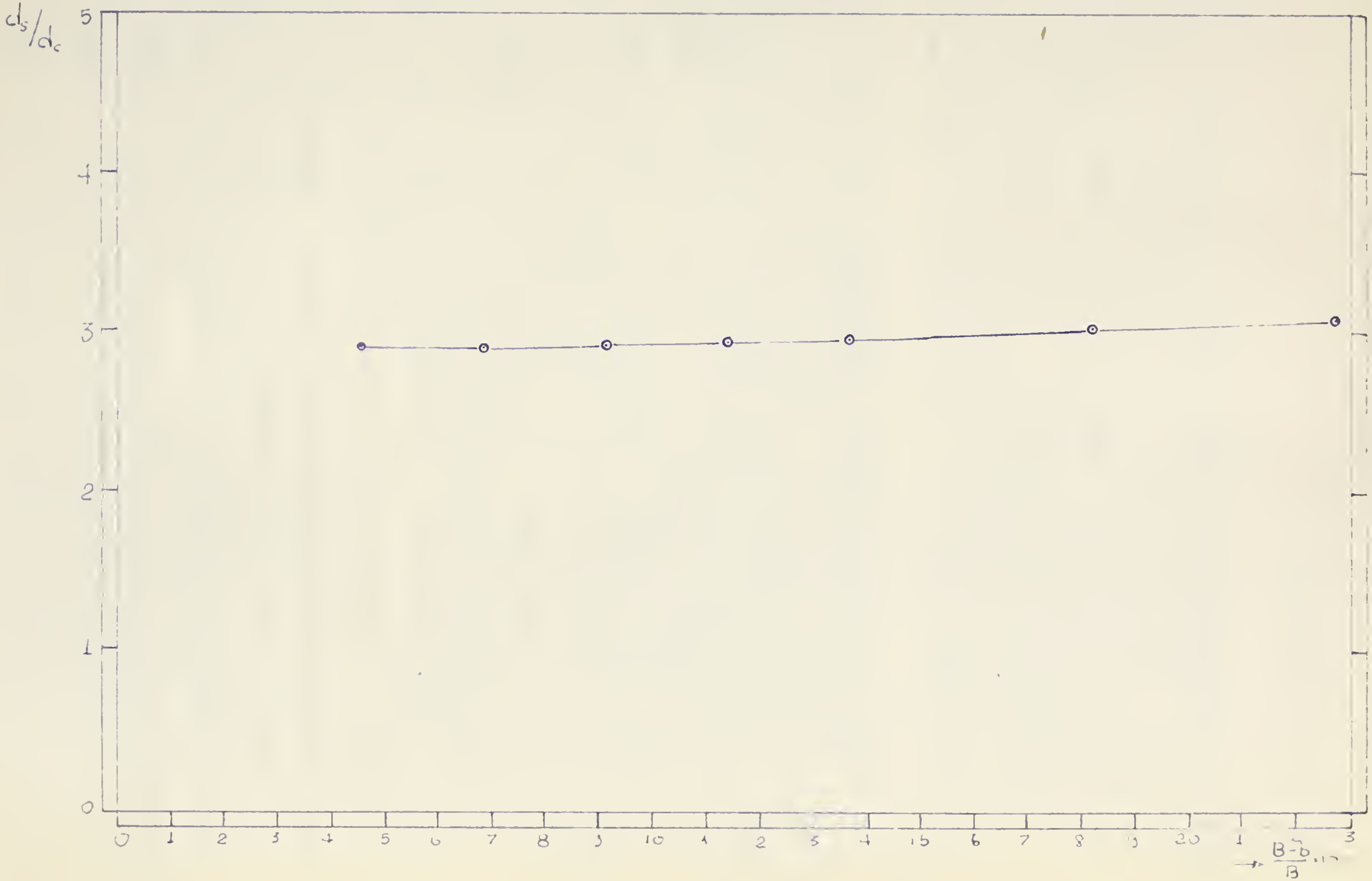

FIG 27. EFFECT OF FLOW CONTRACTION ON MAX. SCOURED DEPTH

EFFECT OF FLOW CHARACTERISTICS

(Experiment VII)

2.42 In section 2.21 the flow characteristics were stated as factors contributing to the scour phenomenon. This experiment has been devoted to study the effect on scour of some flow characteristics. Previously the geometrical aspects of the problem have been studied, therefore this experiment will correlate them to the flow characteristics.

2.43. For this experiment a special pier was built of transparent plexiglass material. The pier model was hollow with wall thickness 5 mm and overall size 2.0 x 6.0 inches. The shape of the model was the standard rounded nose, that is it was similar to the model P13 (Table 5 and Fig. 11) except for the different material and the hollow construction. The bottom of the pier was sealed while the top was left open. The outside of the upstream nose was graduated in inches, half inches and quarters. The initial bed reading was at the 10.0 mark which was used as datum for measuring scour depth (d_s'). As the discharge was increased readings of the lower point of the scour surface were obtained by means of a mirror fastened to a stick with a swivel joint and lowered inside the pier, while a flood light illuminated the outside. It is a fact, observed in all previous experi-

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ments, that the lowest point of the scour surface is located in the foremost point of a pier aligned with the flow.

2.44. Depth of flow was obtained from two manometers, (station 83 and 77 section 1.39) as well as directly in the flume by a graduated ruler 1/5 mm thick equipped with a footing as not to sink in the bed. Measurements of depth by the ruler were made at two flume cross-sections five and six feet upstream the pier (four measurements at each section).

2.45. The discharge was raised as shown in Fig. 28. The time intervals were dictated by the time required to establish flow and scour equilibrium and then take the necessary measurements. This experiment was preceded by a pilot experiment, where it was observed that the scour attained equilibrium after each increase of discharge in no measurable time, and then no matter how long the same discharge was run, the scour would remain constant. Therefore the effect on scour of the time element, as mentioned in section 1.23, could not be evaluated.

2.46. At discharges higher than 2.15 cfs dunes formed and travelled the length of the flume. Their passage over the location of the scour was observable as oscillation of the lower point of the scour surface. The double-amplitude observed for this oscillation was 7/16 inches while after the flume was drained dunes of 1 inch height and wave length

between 10 and 30 inches were measured. This shows that passage of dunes does affect the scoured depth but not to the full height of the dune. In the case of oscillating scour depth, the lowest point of the oscillation was recorded as maximum scour.

2.47. Throughout this experiment the water surface was kept parallel to the initial bed, that is the water surface slope was 0.013 throughout the experiment. This was accomplished by observation of the manometer bank and regulation of tailwater by the adjustable weir (Fig. 2).

2.48. An attempt to measure velocity by a laboratory current meter was made in the pilot experiment, but was abandoned as the effect of the bed and sides was too great for the small depth at which the flume was run.

2.49. Equation 1.1 of section 1.23 was rewritten for this experiment as:

$$\frac{d_s}{d_c} = fn\left(\frac{w}{d_c}, \frac{F_{bo}}{g}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.9)$$

where F_{bo} is the bed factor for vanishingly small charge (Ref.1). From regime theory consideration, and results of Inglis, (Fig. 8-6 Ref. 2) it was assumed that the following equation may relate scour to flow characteristics and width of pier:

$$\frac{d_s}{d_{ro}} = \text{constant} \left(\frac{w}{d_{ro}}\right)^{\frac{1}{4}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.10)$$

where d_{ro} is the regime depth corresponding to bed factor F_{bo}, which in terms of discharge intensity is:

$$F_{bo} = \frac{q^2}{d_{ro}{}^3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.11)$$

$$\text{Also } d_c = \left(\frac{q^2}{g}\right)^{1/3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.12)$$

Eliminating q between equations 2.11 and 2.12 yields:

$$F_{bo} = g\left(\frac{d_c}{d_{ro}}\right)^3 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.13)$$

$$\text{or } \quad d_{ro} = d_c \left(\frac{g}{F_{bo}}\right)^{1/3} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.14)$$

Now d_{ro} can be substituted in equation 2.10 by its value from equation 2.14 producing:

$$\left(\frac{F_{bo}}{g}\right)^{1/3} \cdot \frac{d_s}{d_c} = \text{const.} \left(\frac{w}{d_c}\right)^{1/4} \cdot \left(\frac{F_{bo}}{g}\right)^{1/12} \ldots\ldots\ldots\ldots (2.15)$$

or rearranging:

$$\frac{d_s}{d_c} = \text{const.} \left(\frac{w\ g}{d_c\ F_{bo}}\right)^{\frac{1}{4}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.16)$$

Fig. 30 shows the plot of d_s/d_c versus ($wg/d_c\ F_{bo}$) on double-log paper. In the same Fig. 30, Inglis data from Fig. 31 are analysed and plotted along with the author's. Because two

sands of Inglis are fitted by the same line in Fig. 31, his single fitting line there will become two lines in Fig. 30, that is, one line for $D_m = 0.29$ mm and another for $D_m = 1.3$ mm. The lines in Fig. 30 are deliberately drawn with slope $\frac{1}{4}$ which deviates slightly from the slope of 0.28 corresponding exactly to Inglis own fitting line of Fig. 31. This has been done on the assumption that natural laws usually follow simple indices. The author's own data are also fitted by a line of $\frac{1}{4}$ slope.

Inglis' equation for 0.29 mm sand is:

$$\frac{d_s}{d_c} = 1.8 \left(\frac{w\ g}{d_c F_{bo}}\right)^{\frac{1}{4}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.17)$$

And the author's is:

$$\frac{d_s}{d_c} = 1.7 \left(\frac{w\ g}{d_c F_{bo}}\right)^{\frac{1}{4}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.18)$$

Obviously for all other conditions identical points corresponding to different sizes of bed material should not lie on one line when plotted in the coordinates system of Fig. 30. There appears to be some systematic error in one or both of Inglis' sets of data.

Bed factors zero were calculated by the formula (Ref. 1):

$$F_{bo} = 1.9\ (D_m)^{\frac{1}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.19)$$

2.50. In Figure 31, the author's data are plotted on

Inglis' Fig. 8-6, reference 2, as d_s/w versus $q^{2/3}/w$ yielding a line parallel to his. Inglis equation is (Fig. 31).

$$\frac{d_s}{w} = 1.70 \left(\frac{q^{2/3}}{w} \right)^{0.78} \qquad \text{.....................(2.20)}$$

indicating that an exponent of 3/4 may be more proper. The author's line indicates an exponent of 0.72, therefore supporting the 3/4 slope as having some physical meaning. The corresponding equation derived from the author's data is:

$$\frac{d_s}{w} = 1.43 \left(\frac{q^{2/3}}{w} \right)^{0.72} \qquad \text{.....................(2.21)}$$

2.51. Finally in Fig. 32 scoured depth is plotted against Reynolds number and in Fig. 33 d_s/d_{s2} is plotted q/q_2, where d_s and q are scoured depth and discharge intensity while d_{s2} and q_2 are the same for $Q = 2$cfs, in order to correlate this experiment with the previous ones where $Q = 2$cfs was run. In Fig. 29 depth of flow was plotted against discharge intensity giving a straight line with equation:

$$d = 0.525\, q^{2/3} \qquad \text{.............................(2.22)}$$

Meter Rdng.	Q cfs	q cfs/f	d f	A s.f	V f/s	d'_s f	d_s f	d_c f	S
29	1.16	0.316	0.240	0.878	1.32	0.250	0.495	0.146	0.013
40	1.60	0.436	0.299	1.095	1.46	0.275	0.574	0.181	0.013
50	2.00	0.545	0.350	1.270	1.58	0.290	0.640	0.210	0.013
61	2.44	0.665	0.400	1.465	1.66	0.305	0.705	0.239	0.013
70	2.80	0.763	0.440	1.610	1.74	0.315	0.755	0.263	0.013
80	3.20	0.873	0.480	1.758	1.82	0.325	0.805	0.287	0.013
90	3.60	0.980	0.520	1.910	1.89	0.330	0.850	0.311	0.013
1	2	3	4	5	6	7	8	9	10

Table Results of Experiment VII

(continued on next page)

q	F_{bo}	$\frac{w\ g}{d_c\ F_b}$	$\frac{d_s}{d_c}$	$\frac{d_s}{w}$	$\frac{q^{2/3}}{w}$	$\frac{Vd}{\nu \cdot 10^7}$	$\frac{d_s}{d_{s2}}$	$\frac{q}{q2}$
0.463	2.4	15.30	3.390	2.97	2.78	1.780	0.774	0.580
0.575	2.4	12.35	3.175	3.45	3.45	2.450	0.896	0.800
0.667	2.4	10.65	3.050	3.84	4.00	3.110	1.000	1.000
0.761	2.4	9.35	2.965	4.24	4.57	3.730	1.101	1.220
0.836	2.4	8.50	2.875	4.53	5.02	4.300	1.170	1.400
0.913	2.4	7.80	2.800	4.83	5.42	4.900	1.260	1.600
0.986	2.4	7.20	2.730	5.10	5.93	5.320	1.330	1.800
11	12	13	14	15	16	17	18	19

Table Results of Experiment VII

(Continued from previous page)

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[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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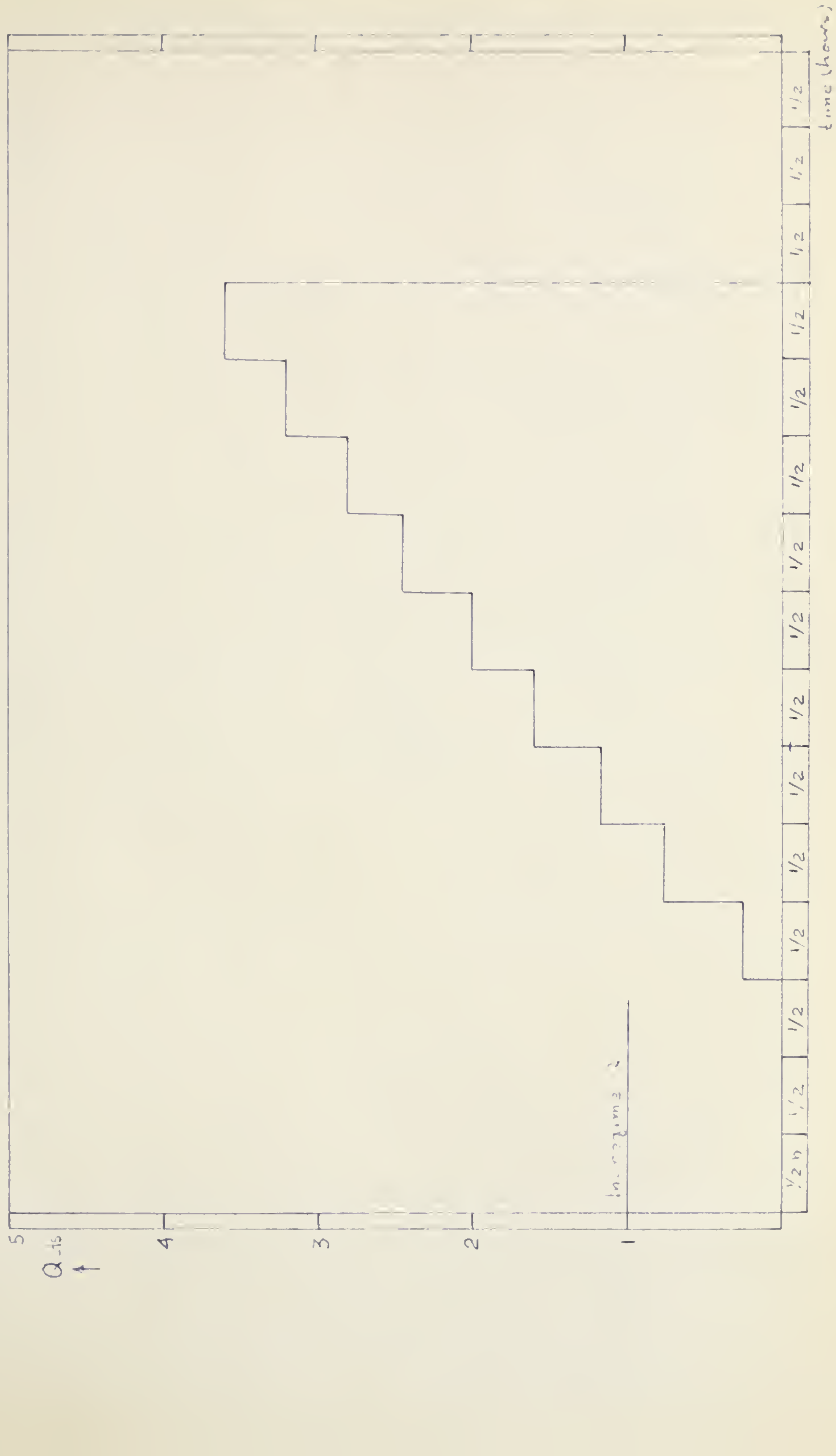

FIG. 28. RATE OF INCREASE OF DISCHARGE (EXPERIMENT VII).

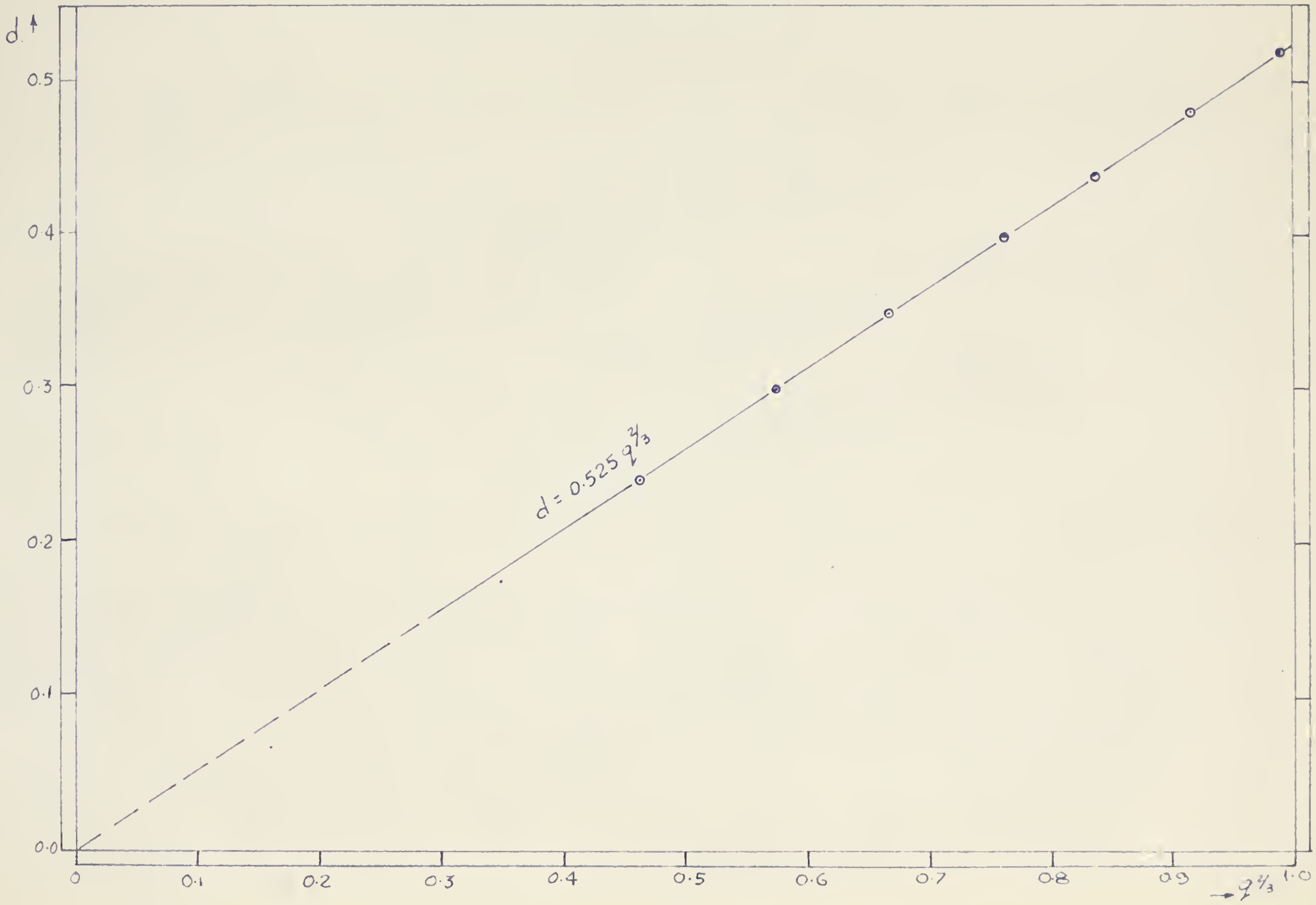

FIG. 29. RELATIONSHIP BETWEEN DEPTH OF FLOW AND DISCHARGE INTENSITY.

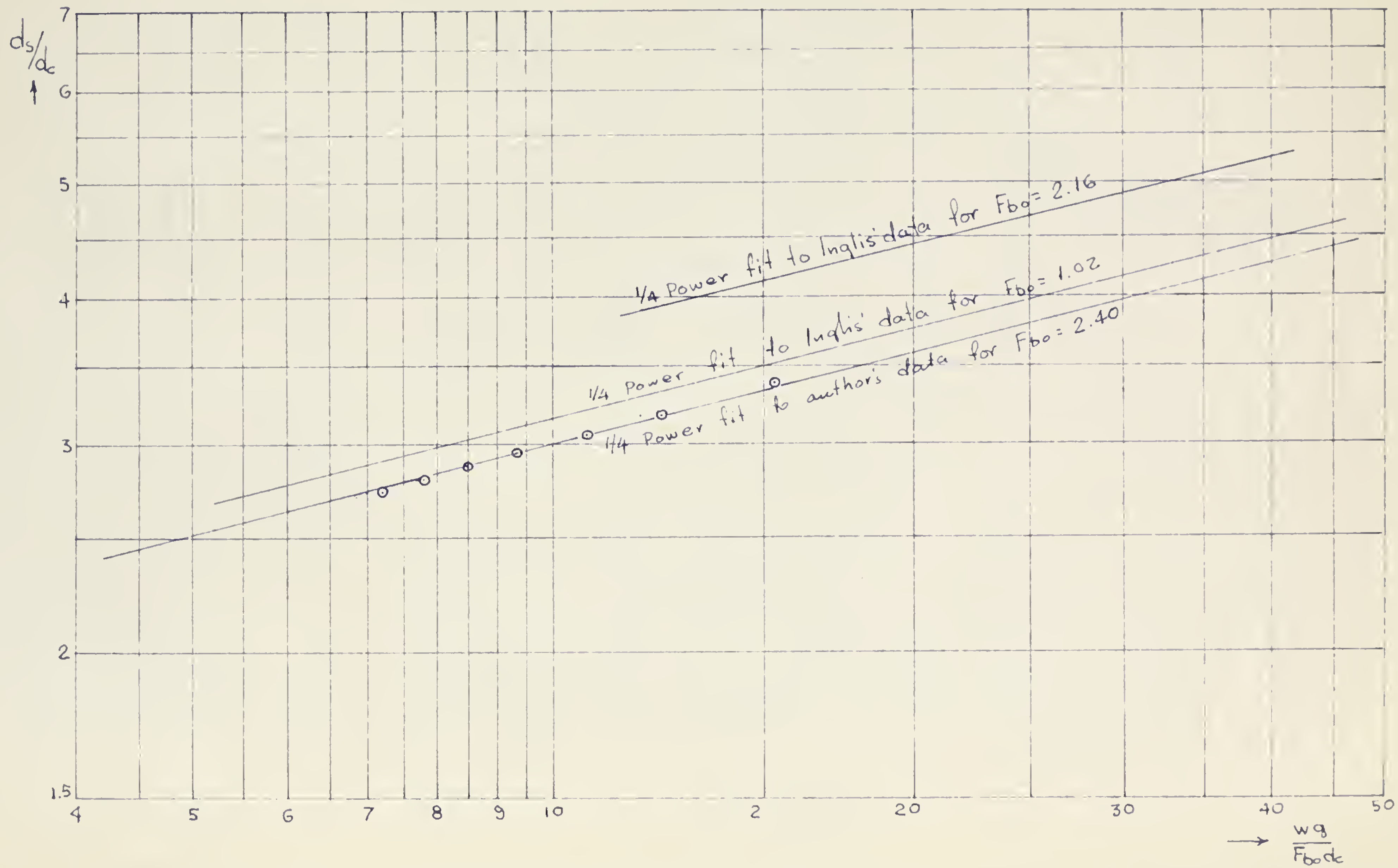

FIG 30. EFFECT OF PIER WIDTH ON MAX. SCOURED DEPTH RELATED TO CRITICAL DEPTH & BED FACTOR

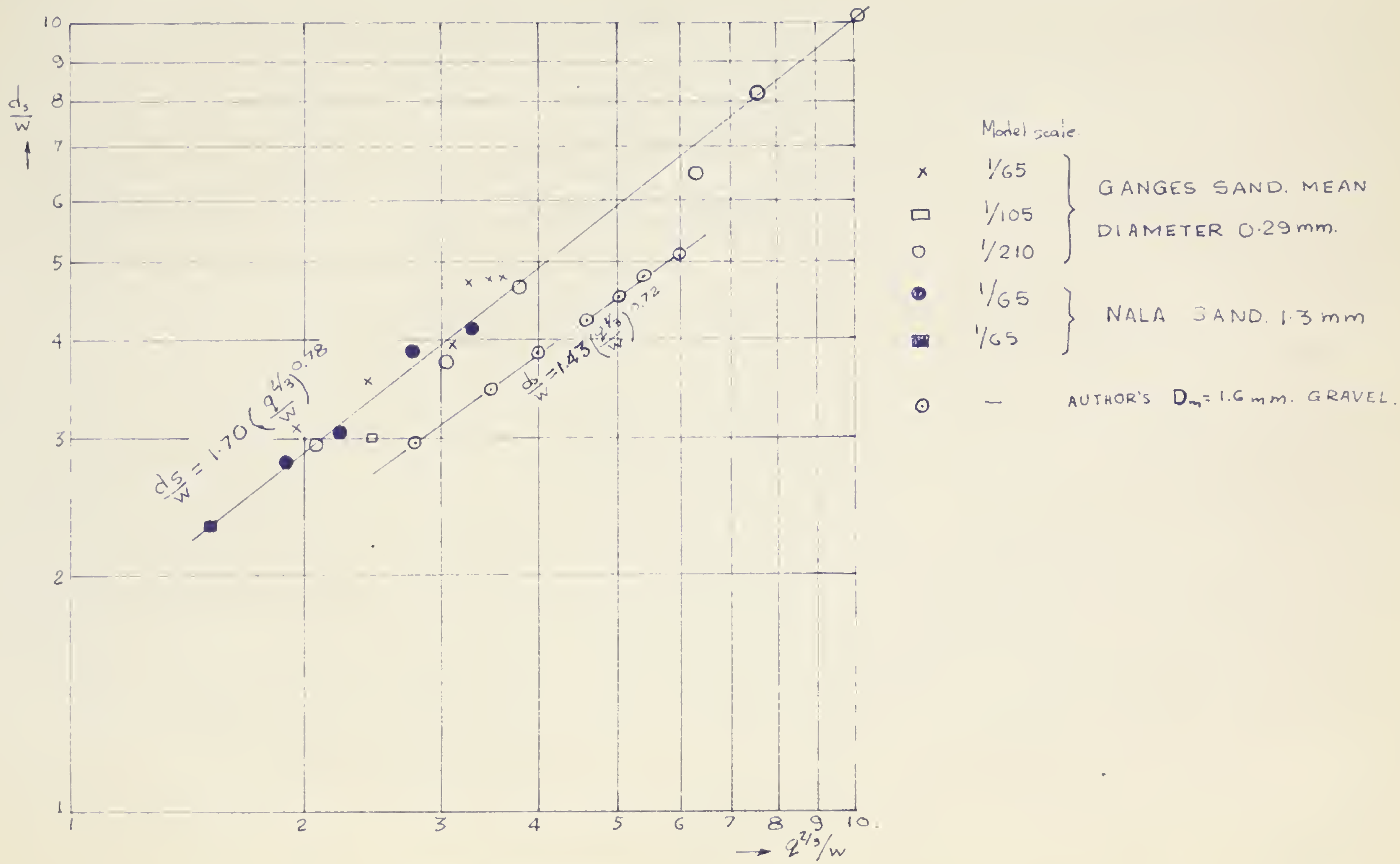

FIG 31. RELATIONSHIP BETWEEN SCOURED DEPTH AT NOSE, WIDTH OF PIER AND DISCHARGE INTENSITY

(AFTER SIR C.C. INGLIS. REFERENCE 2)

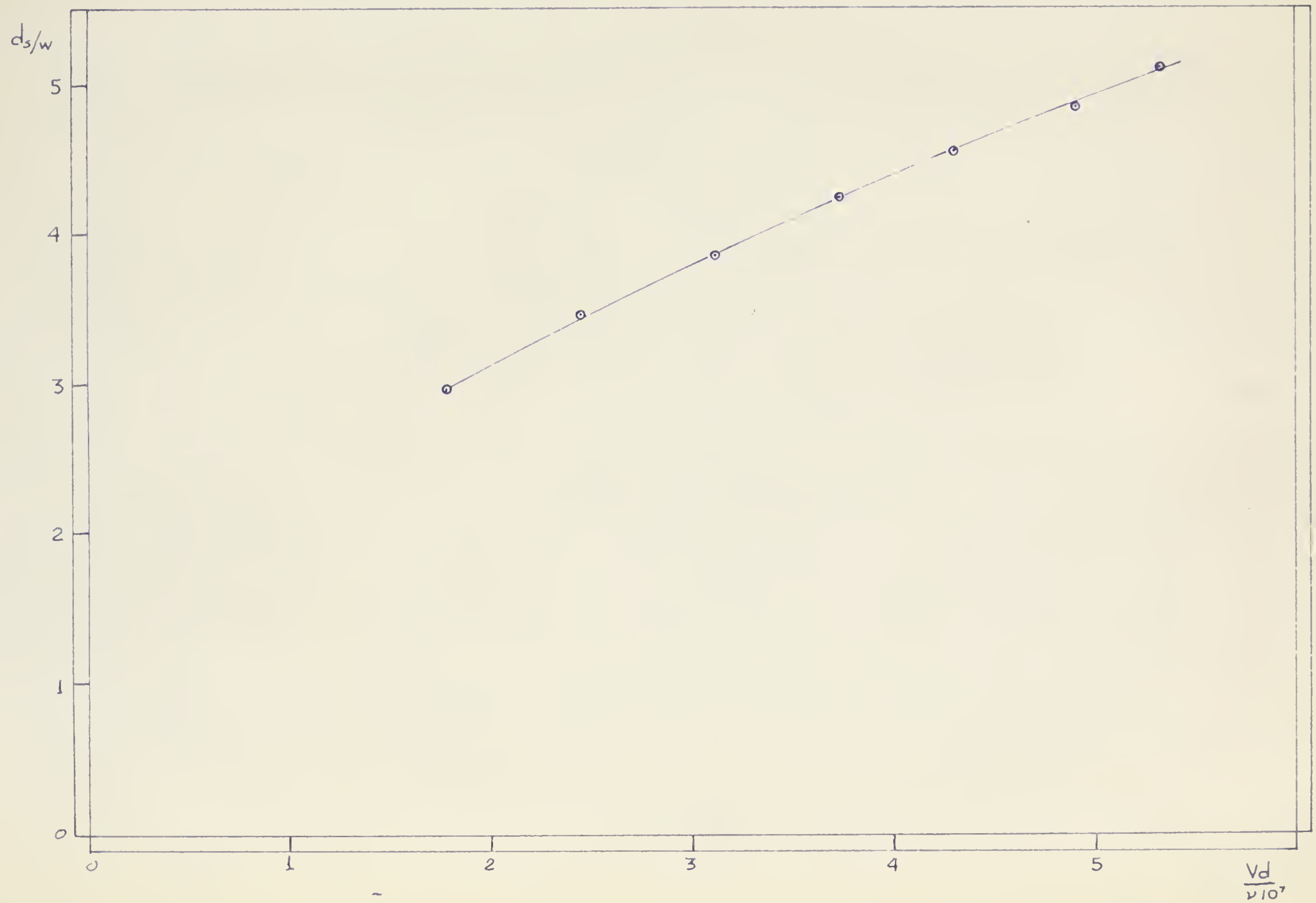

FIG. 32. SCOURED DEPTH VERSUS REYNOLDS NUMBER

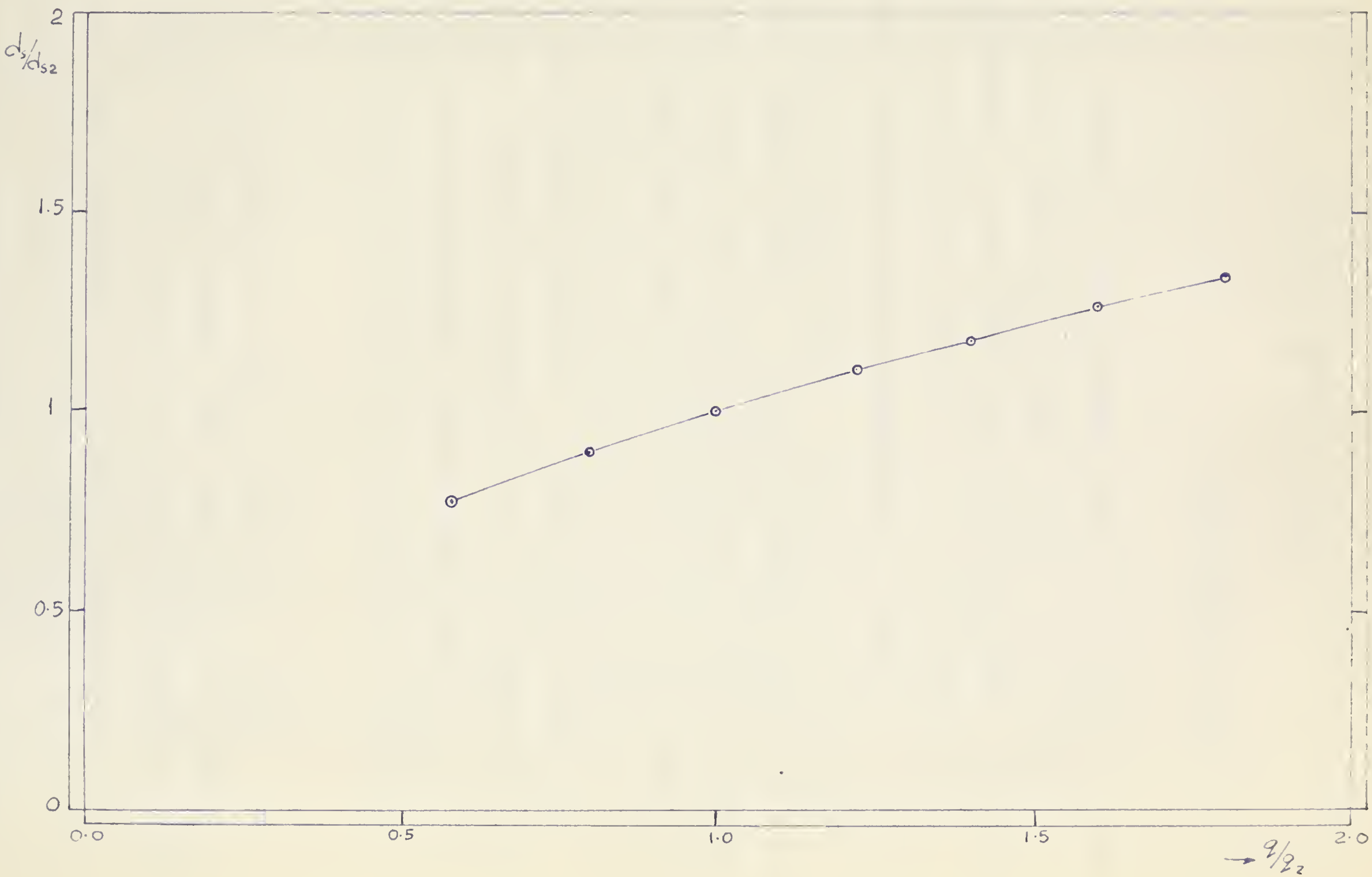

FIG 33. d_s/d_c VERSUS q/q_2 (THE SUBSCRIPT 2 MEANS THAT THE QUANTITY CORRESPONDS TO $G = 2$ c/s).

EFFECT OF FLOWING DEBRIS

(Experiment VIII)

2.52. Rivers do not transport bed load only, but surface load as well, especially at high floods. This is commonly tree branches or even whole trees, and ice cakes. Trees are picked up from burned forests or uprooted by high floods eroding the outside banks of meanders or through the course of cutoffs in forested areas. Ice usually is transported with the spring floods. The water pressure cracks the winter ice cover of the river and big pieces of it flow downstream. Some of this debris, when it hits the nose of the pier, hooks there. As more debris arrive, it is either pushed below or above that already collected. In this manner debris may collect as deep as the bottom of the river around the nose. This is like increasing the width of the pier and all kinds of flow pattern disturbances occur resulting in unforseen scour.

2.53. An attempt was made to study this effect. Four 1" x 6" piers with "rounded ends" were placed at stations 2, 3, 4 and 5. Debris consisting of small bush branches about 4 inches long was placed in increasing quantities in the upstream nose of piers 3, 4 and 5 while pier 2 was left clear. A substantial increase in scour, over that of the clean pier,

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was observed. The results are better shown in the plates of Appendix 3.

2.54. The debris effect is a major one. No quantitative analyses is proposed at present. It is impossible to plot quantity of debris against scour as the pile around the nose takes all sorts of shapes causing all sorts of flow disturbance. And, furthermore, there are no ways for estimating the debris load of a river. It will remain a problem for the experienced river engineer.

CONCLUSION AND SUGGESTIONS

2.55. Experiment VIII was the last one on scour study. A total of forty-four pier models were tested and the results analyzed. A brief summary of the yields is present here together with suggestions that the author feels may orientate further investigators on the subject.

2.56. Experiment I shows that the shape of pier has serious effect on the magnitude of scour. Future investigation into the matter should be toward pier shapes of different cross-sections at different elevations.

2.57. Experiment II shows that increase in pier length does not affect scour seriously. Long piers may be considered as flow dividers rather than obstacles.

2.58. Experiments III and IV show that one of the most important factors on scour development is the width of the pier. It seems that the simple formulas found will change when the waterway is constricted appreciably; this merits investigation.

2.59. Experiment V shows that the angle of attack is of major importance as factor contributing to scour development. It was shown that scour due to this factor is not correctly predicted by using projected width in place of width of pier.

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Still it seems that there is a relationship between projected width and width considered as scour producing factors. Therefore experiments with different length piers, and same width, may prove of value.

2.60. Experiment VI shows that the effect of flow contraction is of negligible magnitude up to the value of 10%, but it suggests investigation of scour around abutments.

2.61. Experiment VII shows the effect of flow characteristics on scour, correlated to pier width which was proven a serious factor, and indirectly to angle of attack. Further experiments on the matter are necessary especially with different gravels, high bed load charges, and broader range of discharge intensities.

2.62. Finally, Experiment VIII shows that debris affects scour tremendously. It is difficult, if not impossible, to do quantitative research on the subject, especially since it is difficult to assess the amount of debris that a river may carry. This will always be left to the experienced river engineer.

2.63. Generally there is an urgency for field observations to supplement laboratory results, as well as to show the value of the experimental results.

CHAPTER 3 APRONS

3.1. The experiment presented in this chapter is a study of the effectiveness of loose stone aprons as means of protecting river bed around bridge piers against scour.

3.2. As outlined in sections 1.6 and 1.7, after an estimate of the anticipated scour around a pier has been done the engineer is concerned with the problem of economical comparison between deep foundation and protection against scour. The most common method of protection is laying loose stone aprons around the pier. The main advantage of this way of protection is the fact that when, and if, the bed starts scouring the loose stone settles instead of leaving voids.

3.3. Fourteen different aprons were studied using stones of three different sizes, as well as their combination. Aprons were laid in various plan shapes as well as different vertical cross sections. The discharge was raised as shown in Fig. 28. Further, it was raised up to 4 cfs and run for four hours. If aprons were still holding, surge waves were introduced at random by closing and opening the valve in an attempt to study the aprons under the most severe conditions.

3.4. The pier models employed in this experiment were

1.0 x 6.0 inches in overall size with the standard rounded nose shape.

STONE APRON MATERIAL

3.5. The material used for laying protective stone aprons around the model piers was sieved between two adjacent standard sieves from natural river gravel, well rounded. Stones of three sizes were separated and used; they will be referred to as Large, Medium and Small material henceforth. The sizes of the stone are given as the diameter of a sphere of the same volume - or weight - as the average stone, and having the same specific weight.

3.6. The numerical values of the stone properties are given in Table 13 together with their ratio to same of bed material. The procedure followed in determining those quantities was as follows: a graduated glass was weighed empty, then a certain number of stones was placed in it and weighed again. After that the glass was filled with water (68°F) and weighed. Therefore the specific weight of stone material, weight and volume of the average stone and equivalent sphere diameter could be determined.

	Bed	Small		Medium		Large	
	Bed Material	Small Stone	Ratio and Bed.Mat.	Medium Stone	Ratio to Bed.Mat.	Large Stone	Ratio to Bed.Mat.
Number of particles tested	not counted *	600	-	500	-	400	-
Aver. weight of each stone (grams)	0.0055	0.283	51	0.713	130	2.70	490
Aver. vol. of each stone (cm^3)	0.00215	0.108	50	0.262	122	1.00	465
Specific weight	2.58	2.63	1.02	2.72	1.055	2.72	1.055
Equiv. sphere diameter (mm)	1.60	5.90	3.69	7.94	4.96	11.40	7.12
1	2	3	4	5	6	7	8

* For bed material the average size, by weight, from the sieve analysis plot was taken as the equivalent sphere diameter.

Table 13. Characteristics of stone apron material

DESCRIPTION OF APRONS TEST

3.7. Fourteen aprons, identified as A1, A2 ....A14, were tested. They are illustrated in Figures 34, 35 ...47 respectively. In experimenting they were placed at the flume in groups as follows. First, A1, A2, A3, A4, A5, A6. Second,A7, A8, A9, A10, A11, A12. Third, A7, A8, A9, A10, A13, A14; that is, in the third group the undamaged aprons of second group were left in place. Further details on the aprons and their testing is given below.

3.8. Aprons A1, A2, A3, A4, A5, A6. These were laid as one layer of large stones, flush with the initial bed and in different plan views as shown in figures 34, 35, 36, 37, 38 and 39. They were subjected to 4 cfs discharge (4 times the regime discharge) for 4 hours. The only displacement of apron material occurred through uplift of some material from the bed beneath the aprons where they touched the up-stream noses of the pier; this caused the aprons to settle an amount equal to stone size for a distance from the piers equal to about 2 stone sizes. Introducing surge waves still did not destroy the apron but the aprons settled further around the noses of the piers to a total of twice the size of the stone; there was no settlement beyond about 3 stone sizes from the piers.

3.9. Aprons A7 and A8 were laid flush with the initial

bed using medium stones in one and two layers, respectively, and the previous experiment repeated. The one layer apron (A7) showed settlement around the nose of the pier but the two layer apron (A8) withstood the 4 cfs flood as well as surge wave severities without showing any sign of fatigue.

3.10. Apron A9 was the same as A7 in plan view, but instead of being flush with the bed, it was exposed, that is laid on the bed. Therefore it was extending one stone size above the bed. The material was of the medium size.

At 1.6 times the regime discharge - that is $Q = 1.6$ cfs - the apron material had been agitated but it remained in about the same place where it was laid. At 2.4 times the regime discharge the apron could be said to be destroyed in the sense that stone had been carried away and deposited downstream of the pier. The small amount of stone remaining had settled around the pier at a depth of 0.16 ft. or over four times the size of the stone. This prevented the scour to go deeper.

3.11. Apron A10 was constructed with the same layout as Apron A7 and A8, flush with the initial bed but the material was a mixture of the three sizes mentioned above, that is one third large, one third medium and one third small. The material was mixed before the apron was laid. The thickness of the apron was the size of a large stone. It withstood

up to four times the regime discharge but not for too long. After one half hour of attack by the 4 cfs discharge, the largest amount of the small size stone was picked up and deposited downstream of the pier, spread in a four feet reach of the flume. This suggests arrangement of stone in layers.

3.12. Aprons A11 and A12 were constructed flush with the initial bed as shown in Figs. 44,45. The material was small stone and A11 consisted of one layer while A12 was two layers thick. At 1.6 times the regime discharge Apron A11 was completely destroyed; that is, all the material had been picked up by the flow and spread downstream of the pier as far as eight feet. Apron A12, which consisted of twice the thickness of A11, was destroyed around the pier in a pattern similar to those of Figures 19 and 25.

3.13. Finally, aprons A13 and A14 were constructed from small material flush with the initial bed as shown in Figs. 46 and 47. They replaced aprons A11 and A12 in the second group. Although they were far heavier aprons than A 11 and A12 they did not prove more effective. They were destroyed and the stone washed away in the same matter and at the same discharge as aprons A11 and A12.

3.14. Reviewing the above it appears that the best over-all behaviour of the three stone sizes in the range of dis-

charge used was by the medium stone laid flush with the bed in two layers, that is, apron 8. This suggests that stone of five times the average bed material size should be used in laying aprons at the elevation corresponding to this model's bed in order to withstand floods up to four times the regime discharge. Because of the behaviour described in 1.58 the depth of flow in the flume was considerably less than would correspond to a prototype so the model aprons would behave as if set well above prototype bed.

3.15. The overall behaviour of apron A10, that is, the one constructed from mixed size stones, suggests that the smaller stone should be laid in a layer underneath the larger thus sealing and protecting the bed material from being picked up while the layer is protected by the larger stone layers above.

3.16. The testing of aprongA9 shows that aprons should not be laid exposed, that is above the initial bed. In the common case where degradation is expected, they should be laid in such a manner as not to be exposed above bed when it degrades.

3.17. Further research is necessary on the subject. Investigation should be carried out with broader ranges of discharges and charges, as well as sizes and specific weight of stones. Further aprons laid in chicken-wire crated stone are worthwhile investigation for large stone is not available, in economic prices, at every construction site.

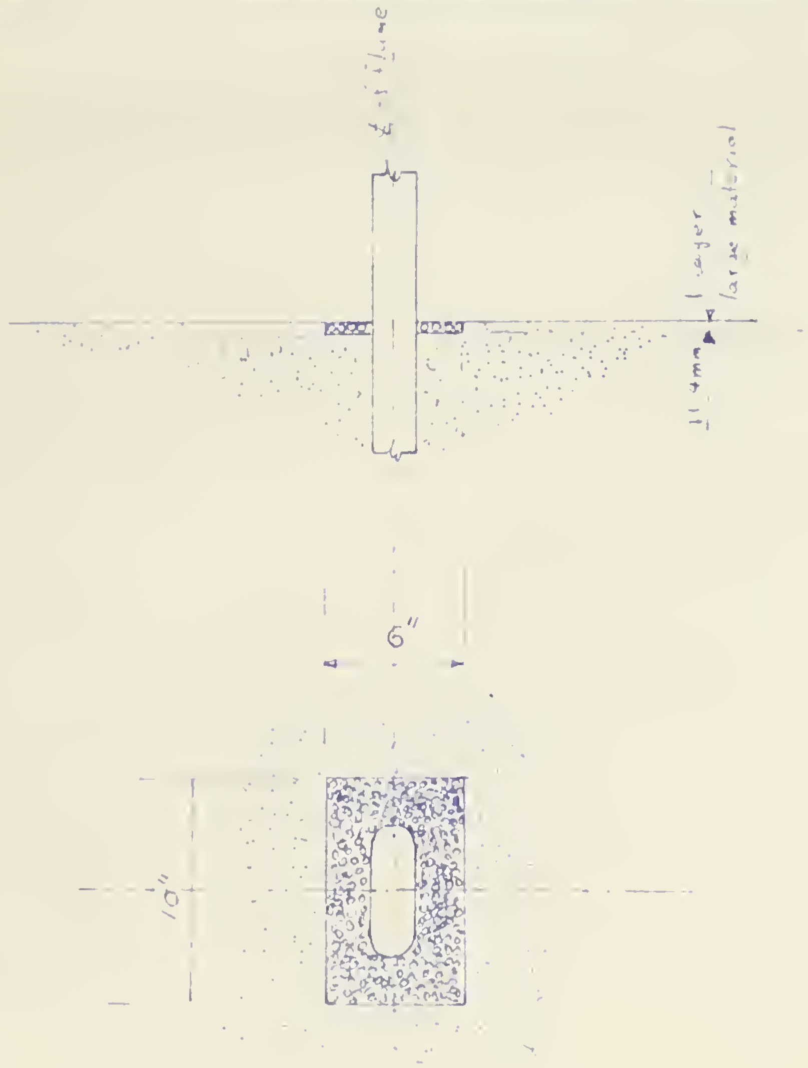

FIG 34 APRON A1 - ELEVATION AND PLAN.

SCALE 1:10

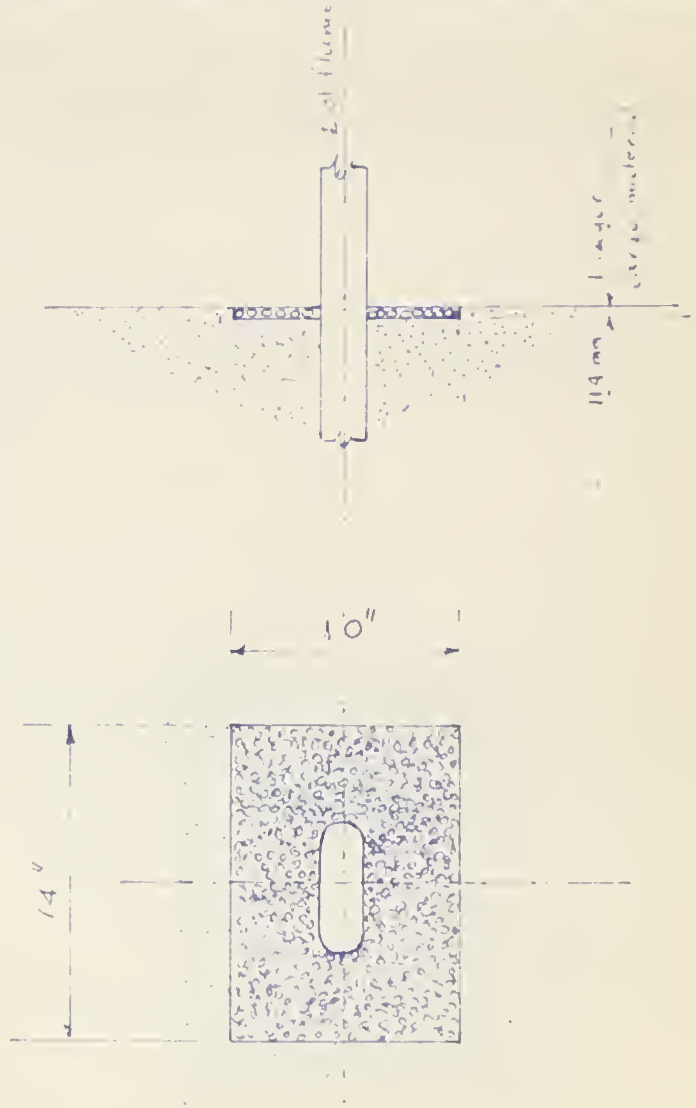

FIG 35. APRON A2 - ELEVATION AND PLAN.

SCALE 1:10

FIG 36 APRON A3 - ELEVATION AND PLAN

SCALE 1:10

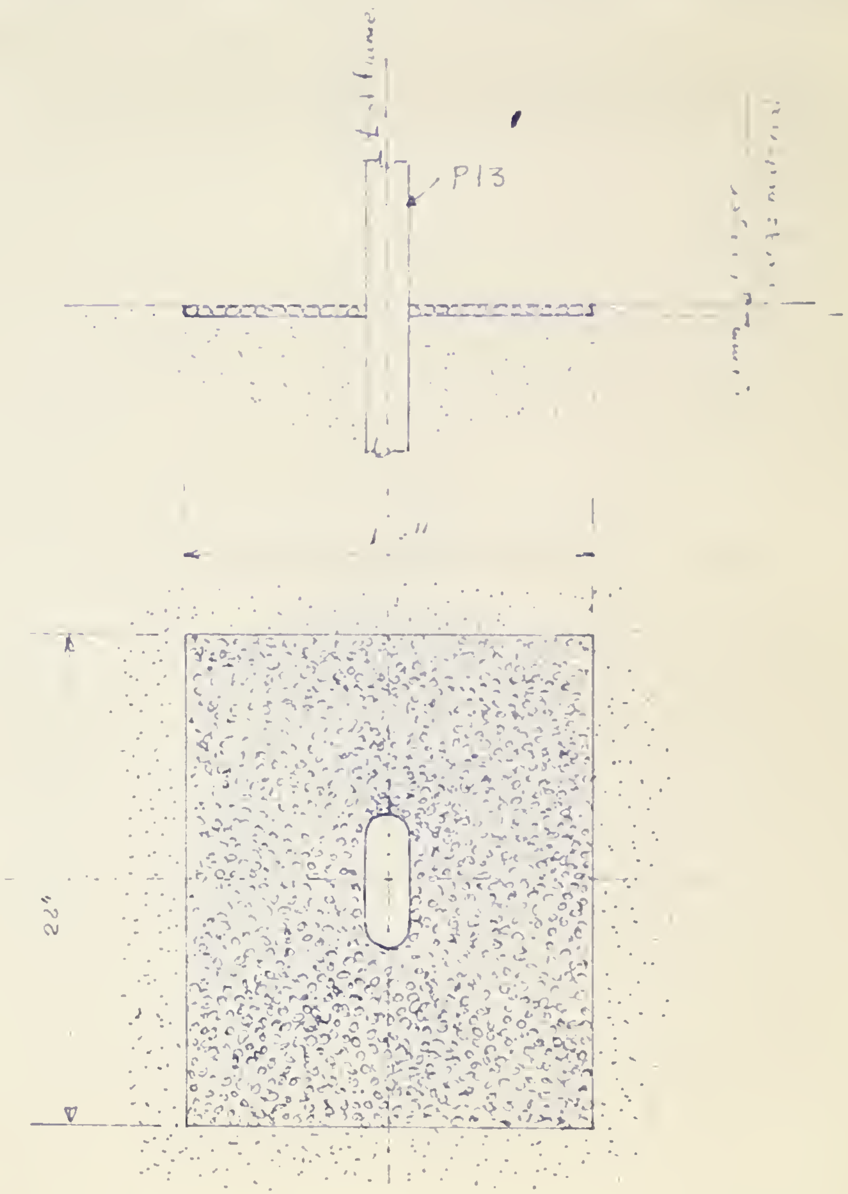

FIG 37 APRON A4 - ELEVATION AND PLAN

SCALE 1:10

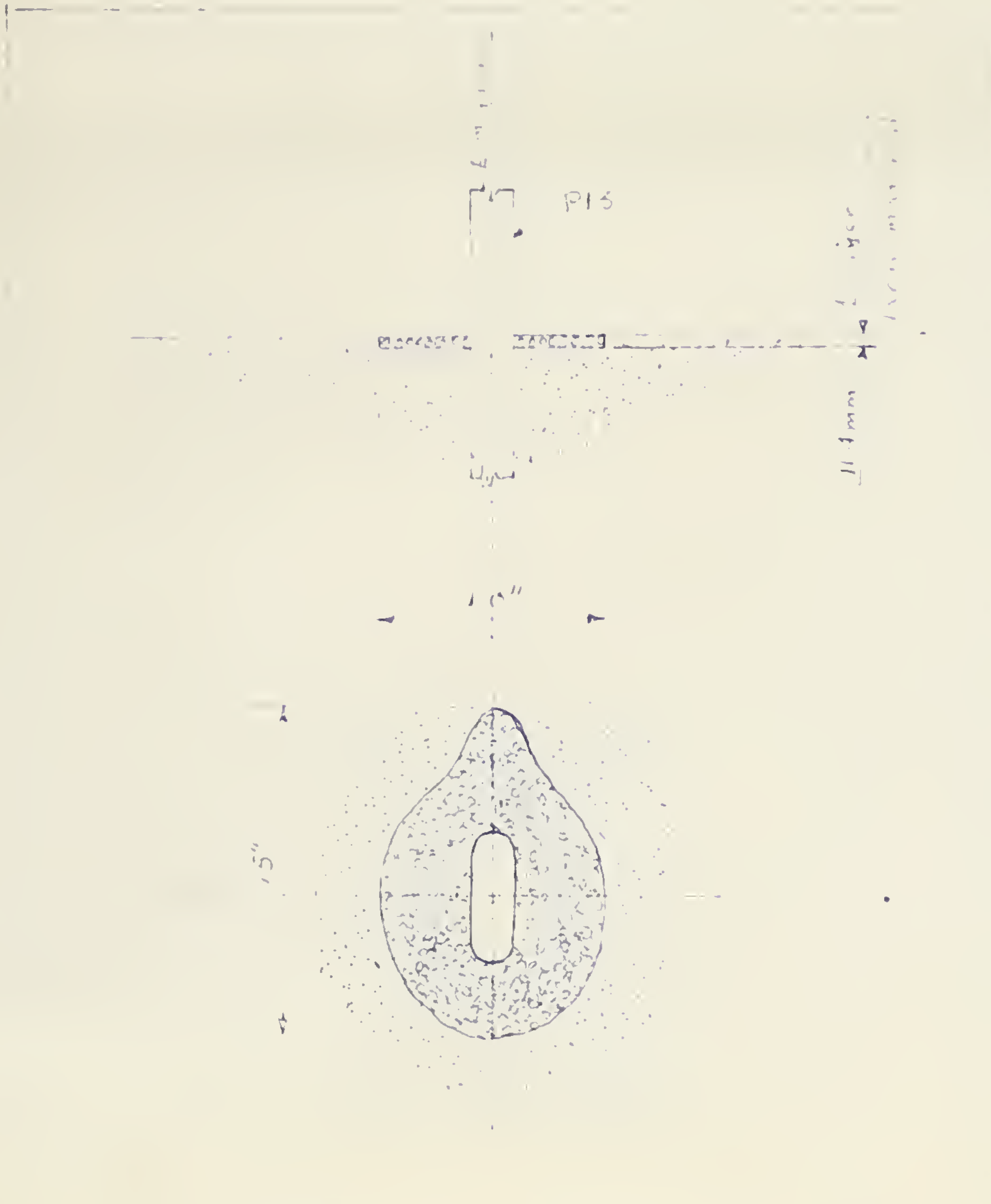

FIG. 38 APRON A5 - ELEVATION AND PLAN

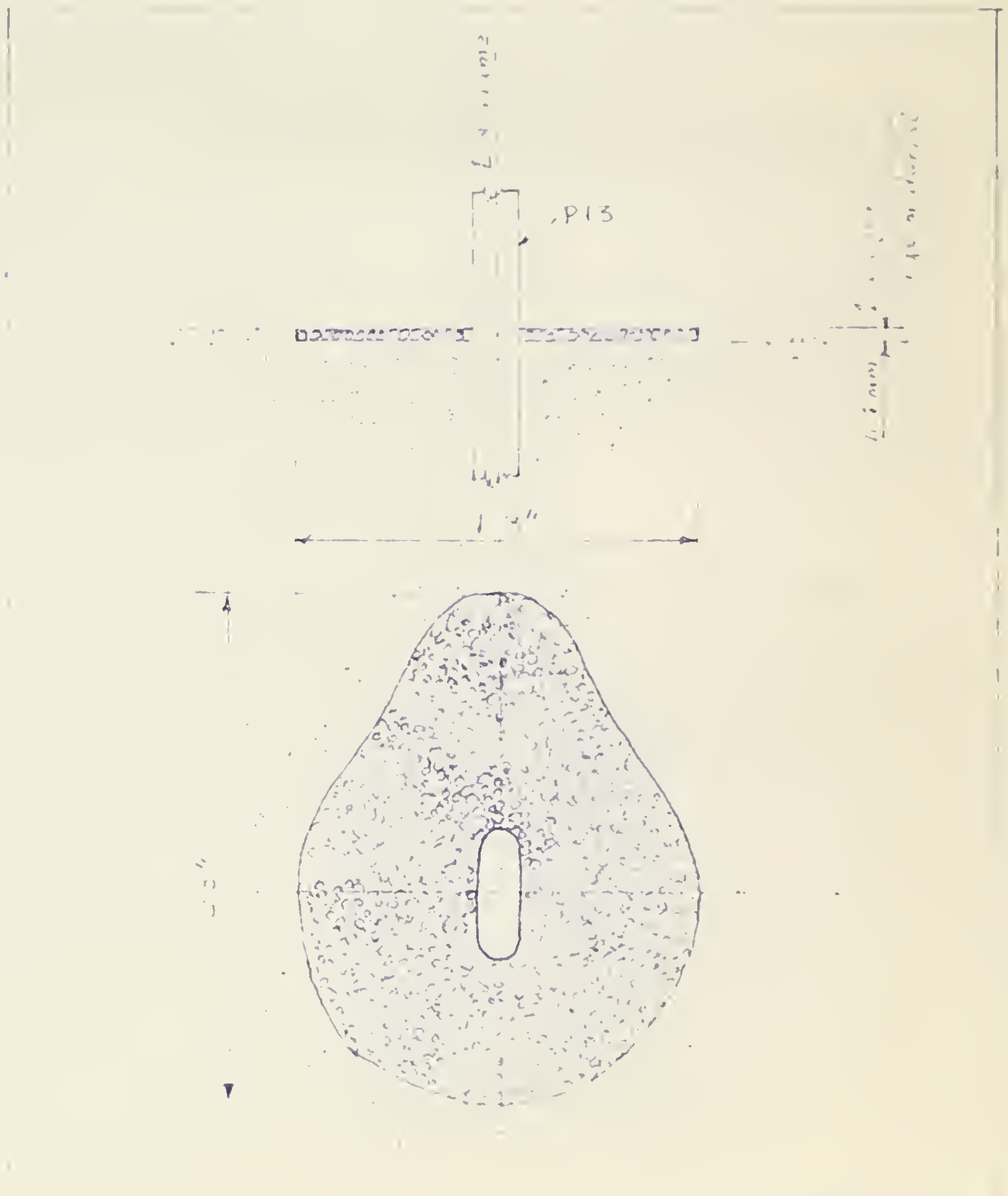

FIG. 39 APRON A6 ELEVATION AND PLAN

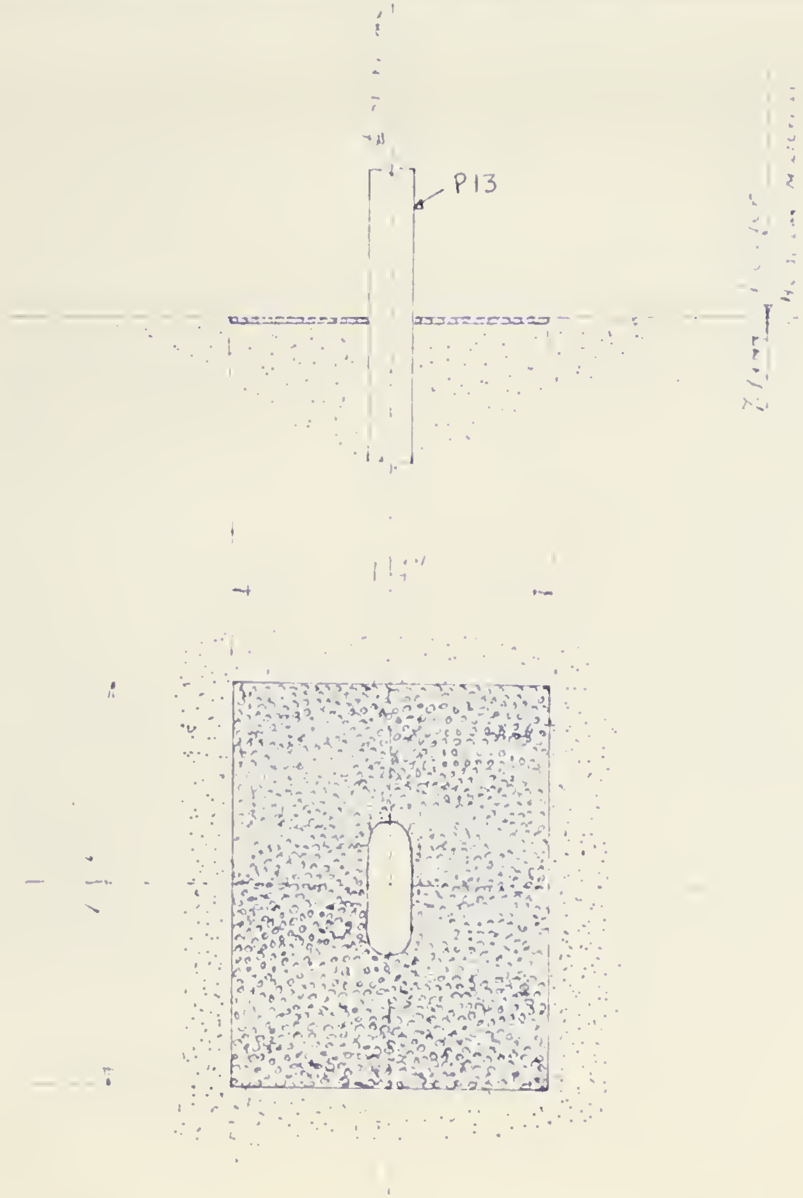

FIG. 40 APRON A7 - ELEVATION AND PLAN

SCALE 1:10.

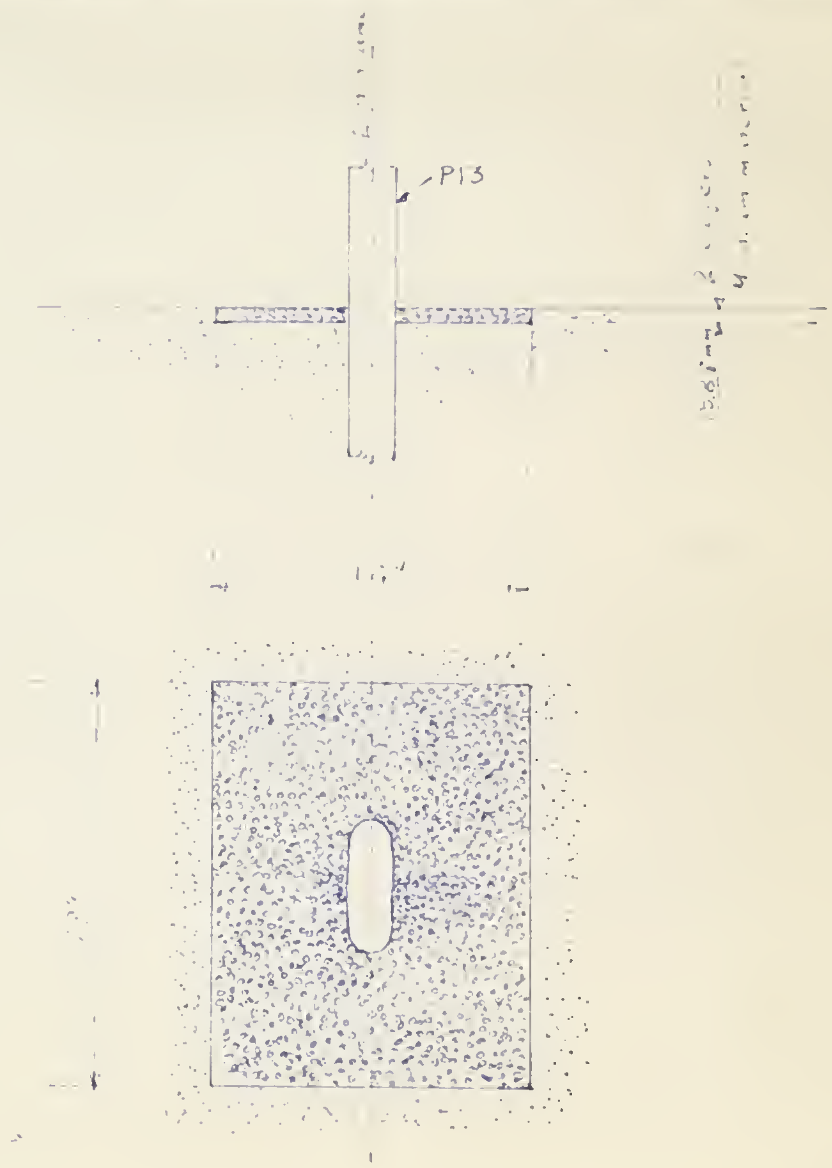

FIG. 41 APRON A8 - ELEVATION AND PLAN

SCALE 1:10.

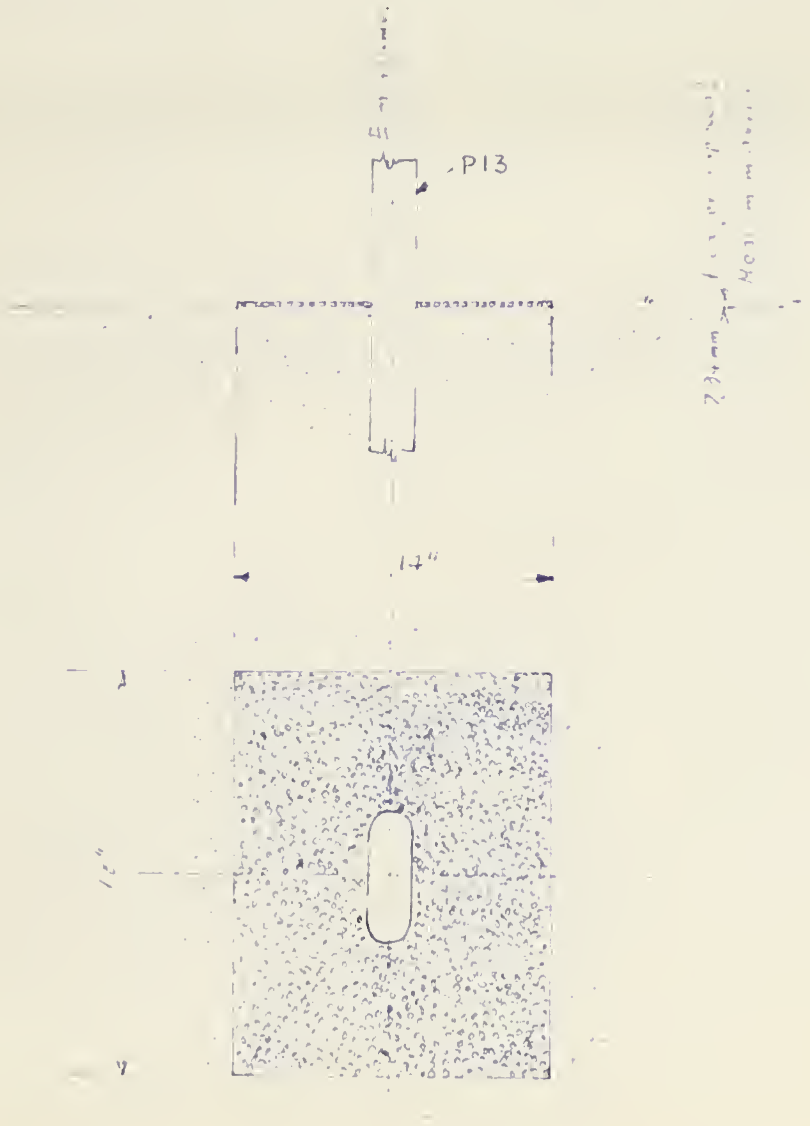

FIG 42. APRON A9 · ELEVATION AND PLAN

SCALE 1:10

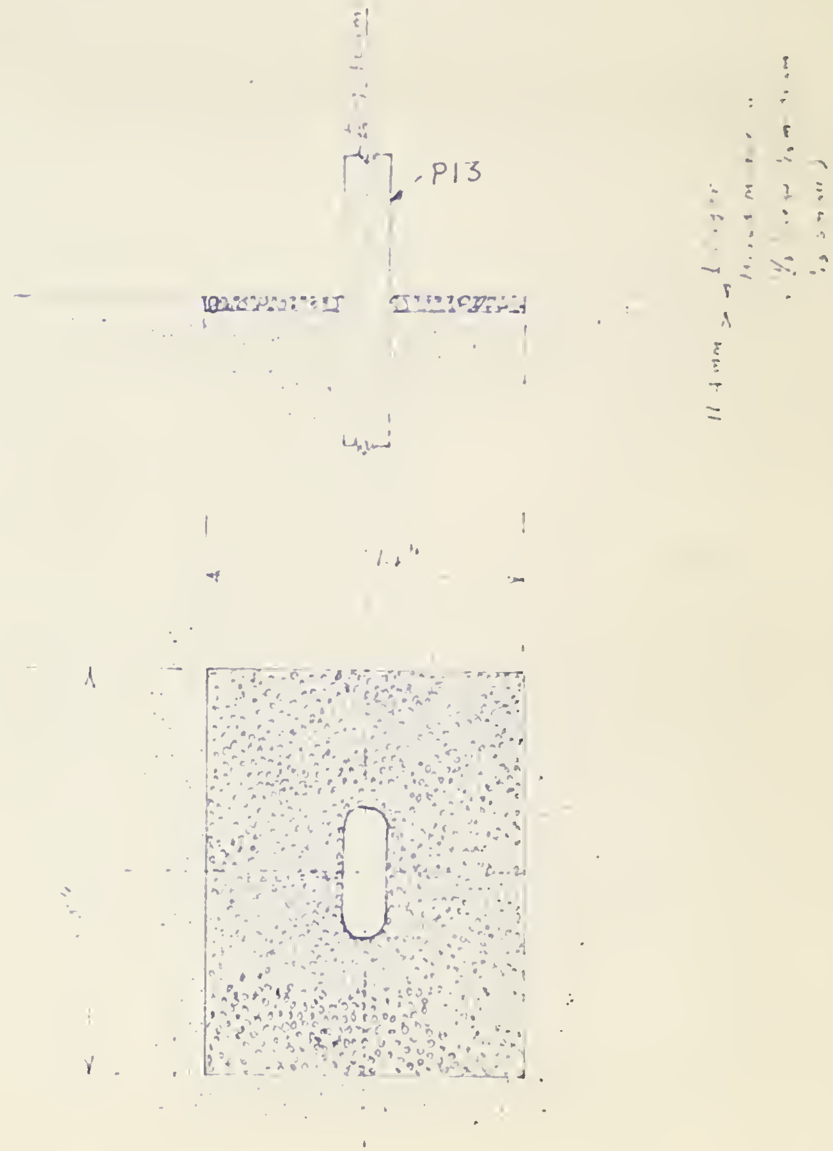

FIG 43. APRON A10-ELEVATION AND PLAN

SCALE 1:10

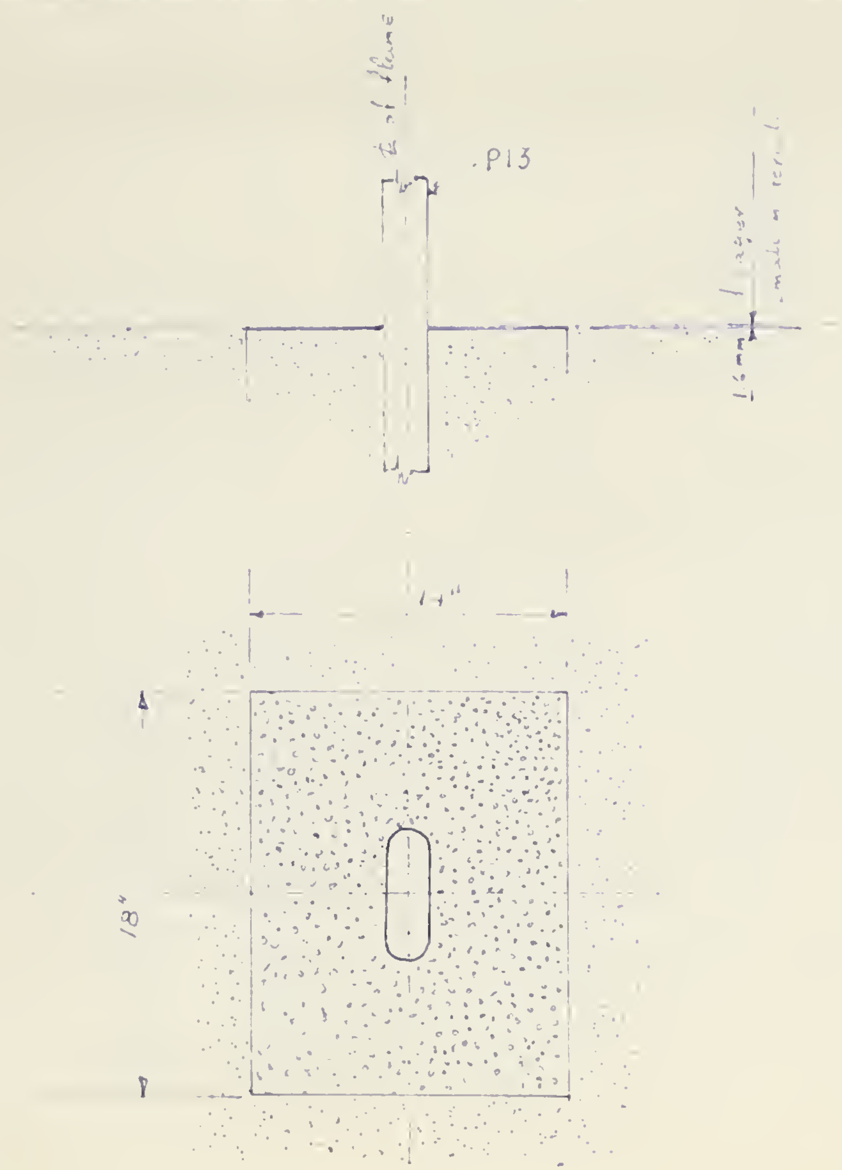

FIG 44. APRON A 11 - ELEVATION AND PLAN

SCALE 1:10

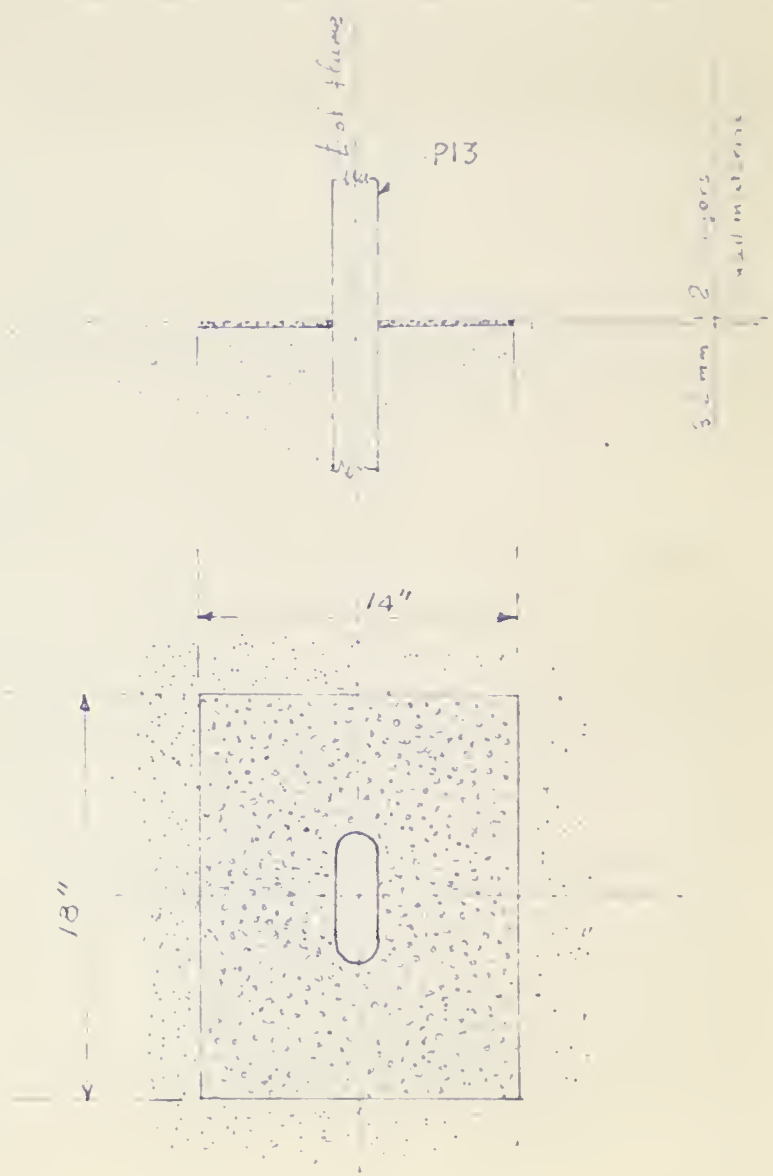

FIG 45. APRON A 12 - ELEVATION AND PLAN

SCALE 1:10

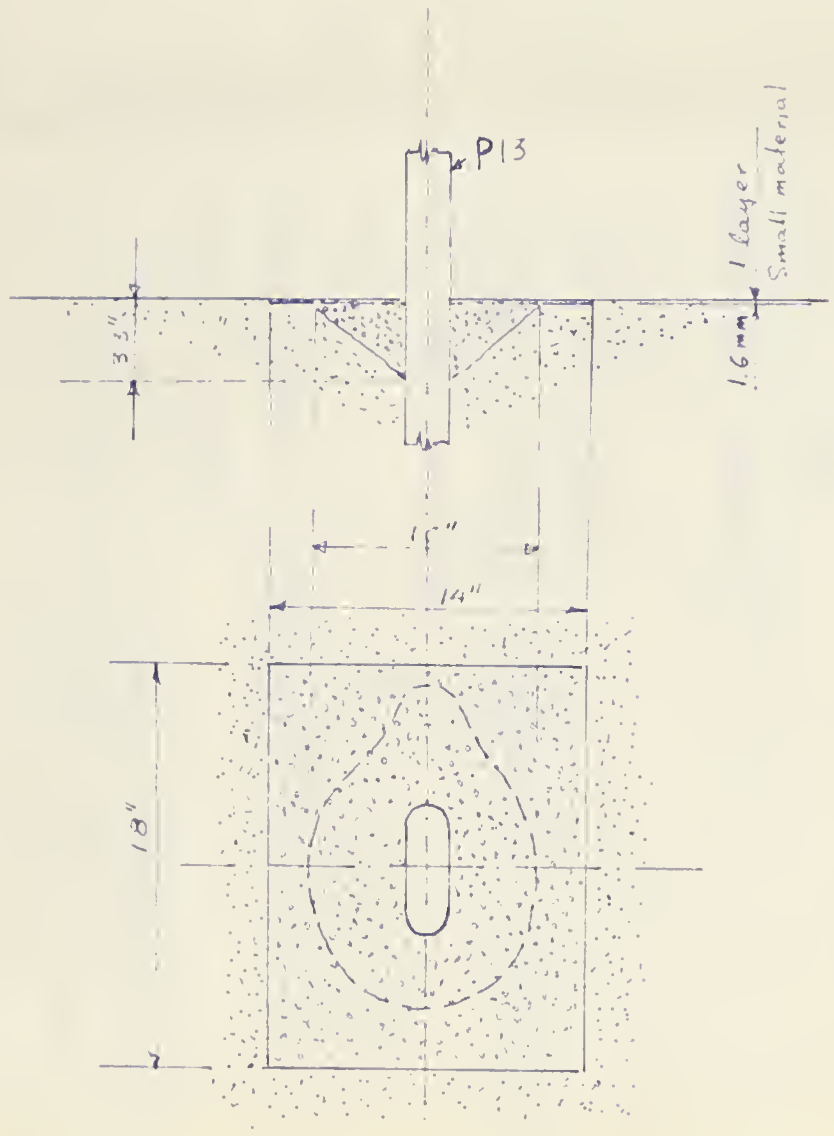

FIG 46 APRON A13-ELEVATION AND PLAN

SCALE 1:10

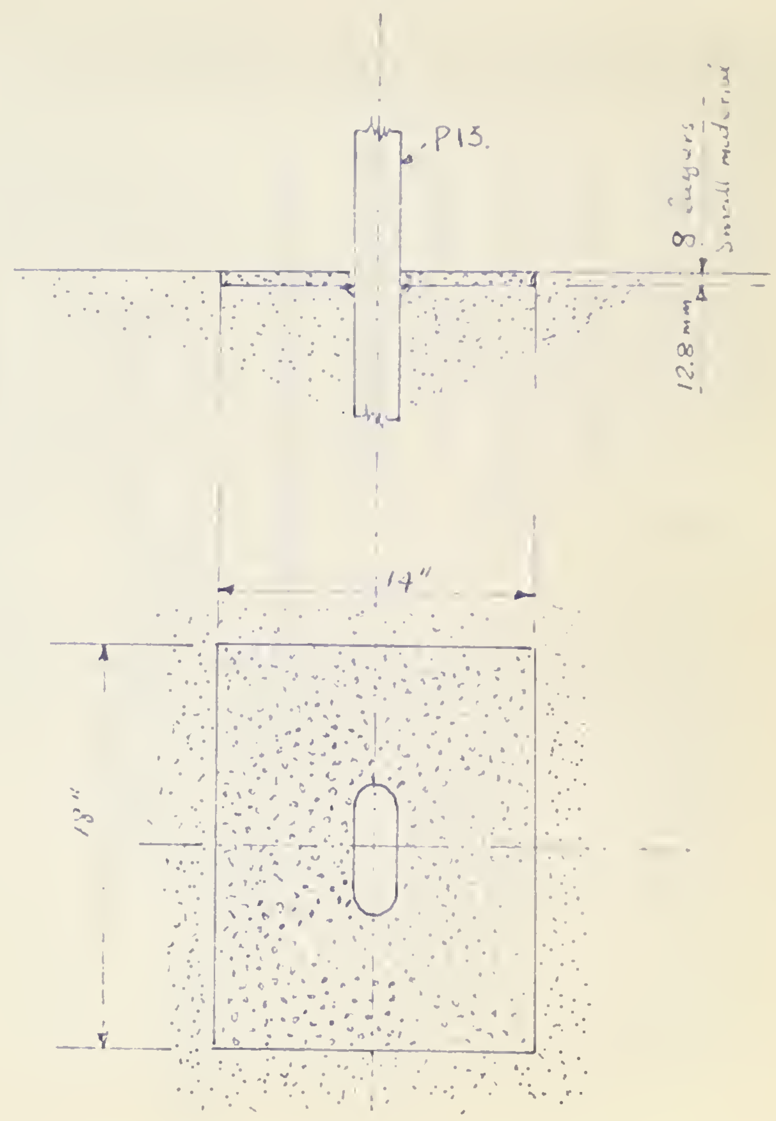

FIG 47. APRON A14 - ELEVATION AND PLAN

SCALE 1:10

APPENDIX I - NOTATION

A	wetted area
a	area of horizontal cross-section of pier.
B	width of flume.
b	width of waterway.
d	depth of flow
d_c	critical depth
d_{ro}	regime depth calculated using F_{bo}.
d_s	scoured depth (measured from water surface).
d'_s	scour depth (measured from bed).
F_{bo}	bed factor zero.
g	acceleration of gravity.
θ	angle of attack of flow.
L	perimeter of horizontal cross-section of pier.
l	length of pier.
ν	kinematic viscosity.
Q	discharge.
q	discharge intensity
q'	discharge intensity referring to restricted waterway.
s	water surface slope.
t	time
V	mean velocity of flow.
V_s	volume of scour.
V_p	volume of pier per linear (vertical) unit (= a)
D_m	mean (in terms of weight) size of gravel.

APPENDIX II

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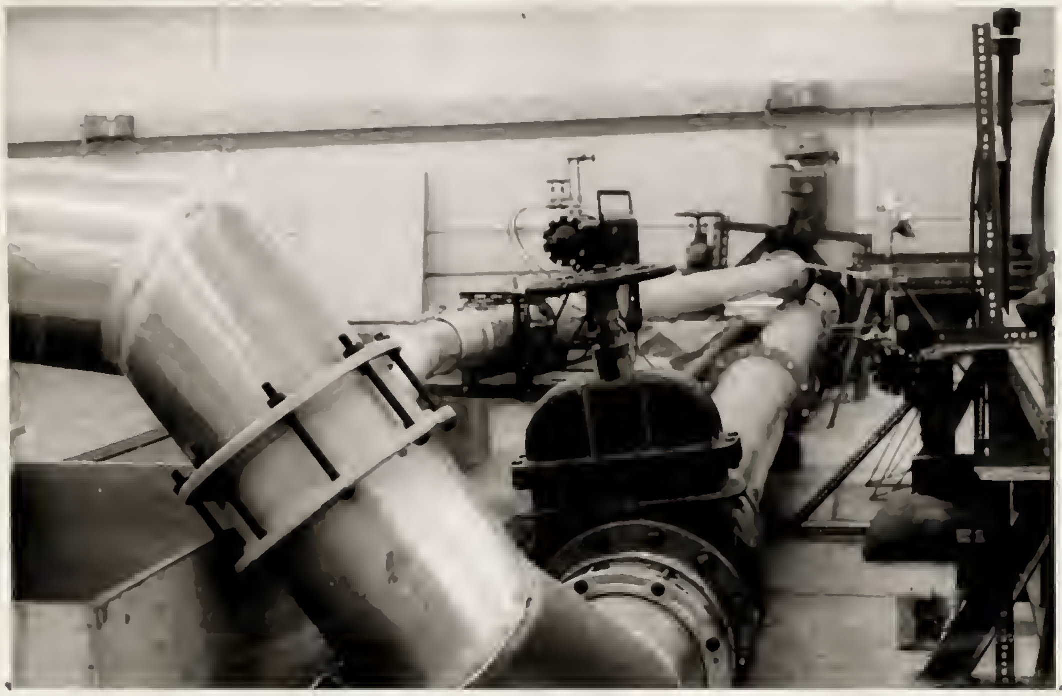
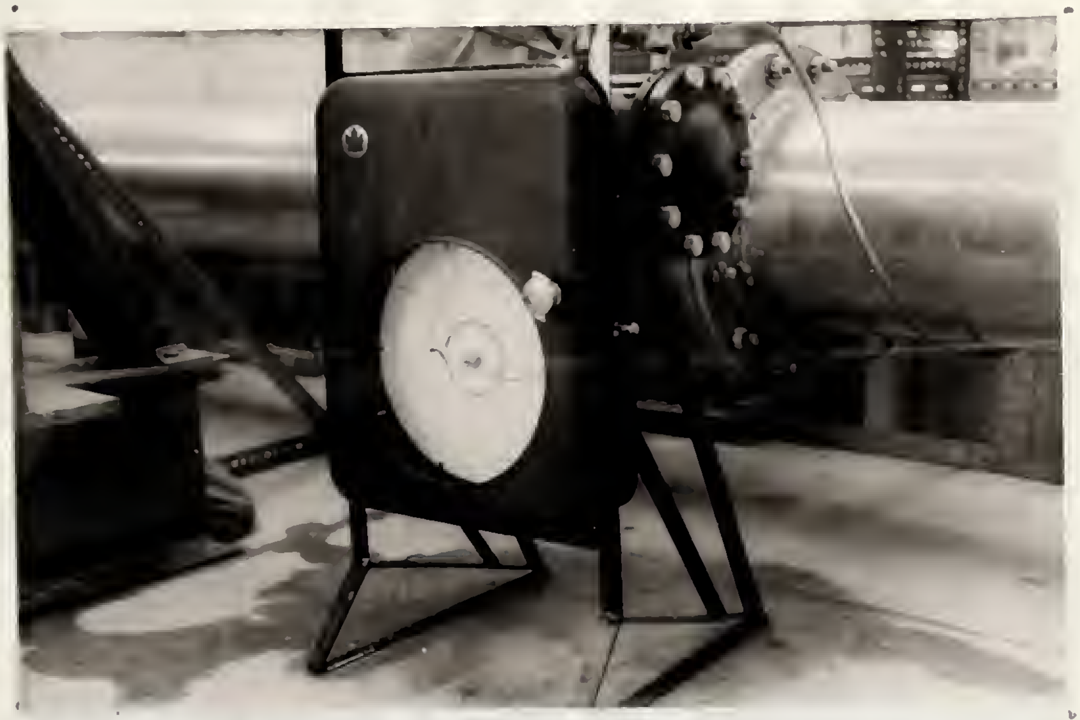
APPENDIX III

PLATES

PLATES 1 & 2 - GRAVEL RIVERS IN ALBERTA





PLATES 3 & 4 - PUMP, WATER SUPPLY LINE & ORIFICE FLOW METER

PLATE 5. ENERGY DISSIPATOR. (Looking D/S)

PLATE 6 SEDIMENT FEEDER

PLATE 7. OVERALL VIEW OF FLUME

PLATE 8. VIEW OF HEADWORKS

PLATE 9. MANOMETER BANK

PLATE 10. ADJUSTABLE WEIR (Flume outlet)


PLATE 11. CONTOURGRAPH


PLATE 12. LAB. CURRENT METER

PLATES 13 and 14. DUNES (Looking U/S)

PLATES 15 and 16 DUNES. (Looking U/S)

PLATE 17. FLOW AROUND LENTICULAR PIER (Exp.I)

PLATE 18. FLOW AROUND ELLIPTICAL PIER (Exp.I)

PLATE 19. FLOW AROUND ROUNDED NOSE PIER (Exp.I)

PLATE 20. FLOW AROUND BEVELLED NOSE PIER (Exp.I)

PLATE 21. FLOW AROUND RECTANGULAR PIER (Exp. I)

PLATE 22. SCOUR AROUND BEVELED NOSE PIER (Exp.I)

PLATE 23. FLOW AROUND 1" x 6" R. N. PIER.(Exp.III)

PLATE 24. FLOW AROUND $4\frac{1}{2}$" x 6" R. N. PIER. (Exp.III)

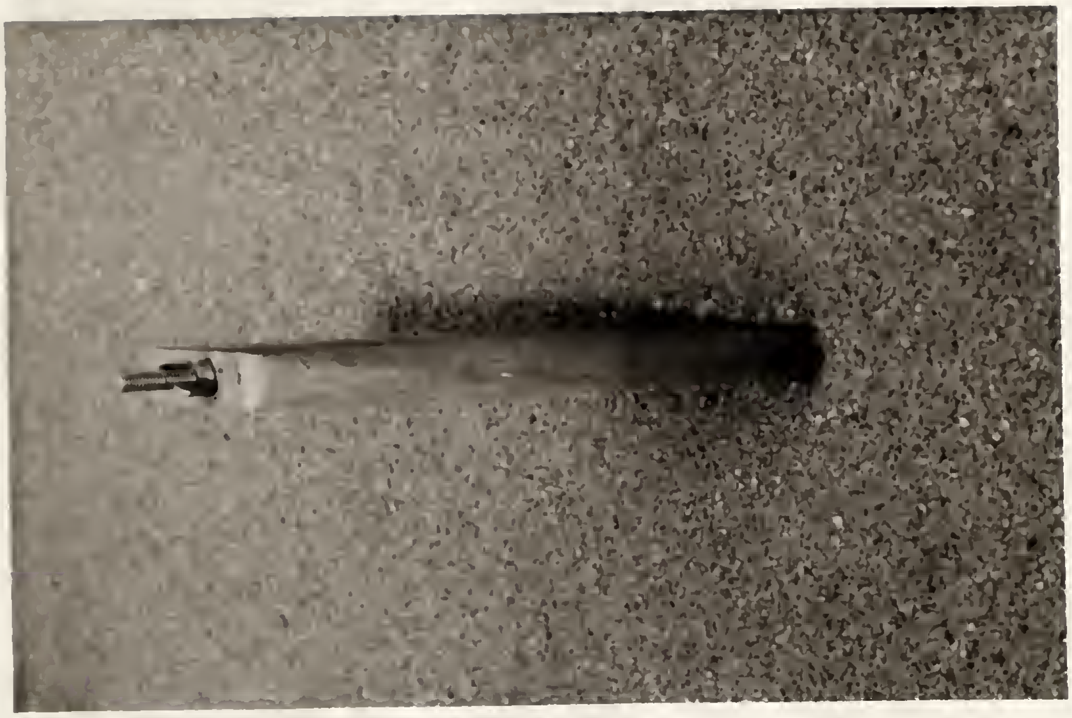


PLATE 25. SCOUR AROUND 1" x 6" R.N. PIER (Exp.III)

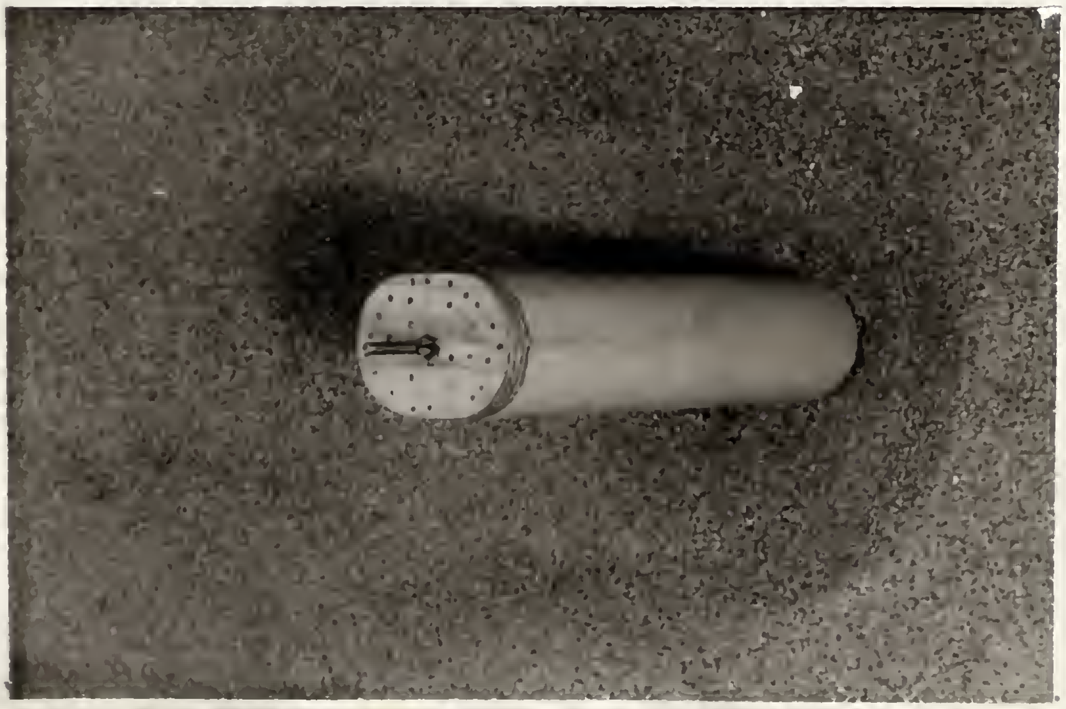
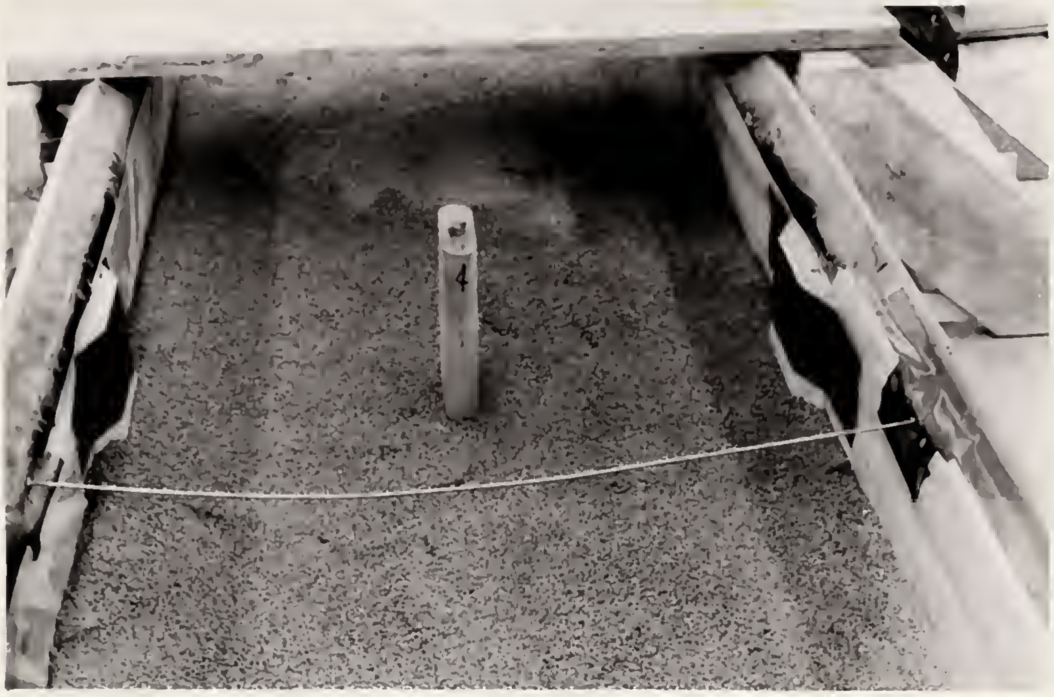
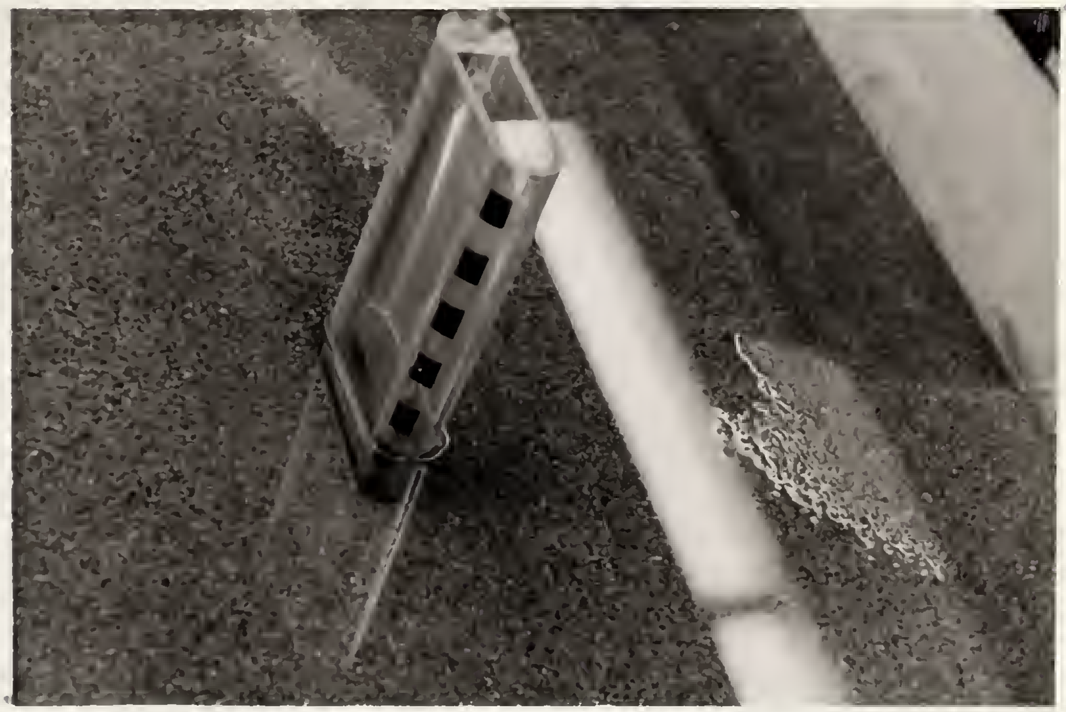


PLATE 26. SCOUR AROUND $4\frac{1}{2}$" x 6" R.N. PIER (Exp.III)



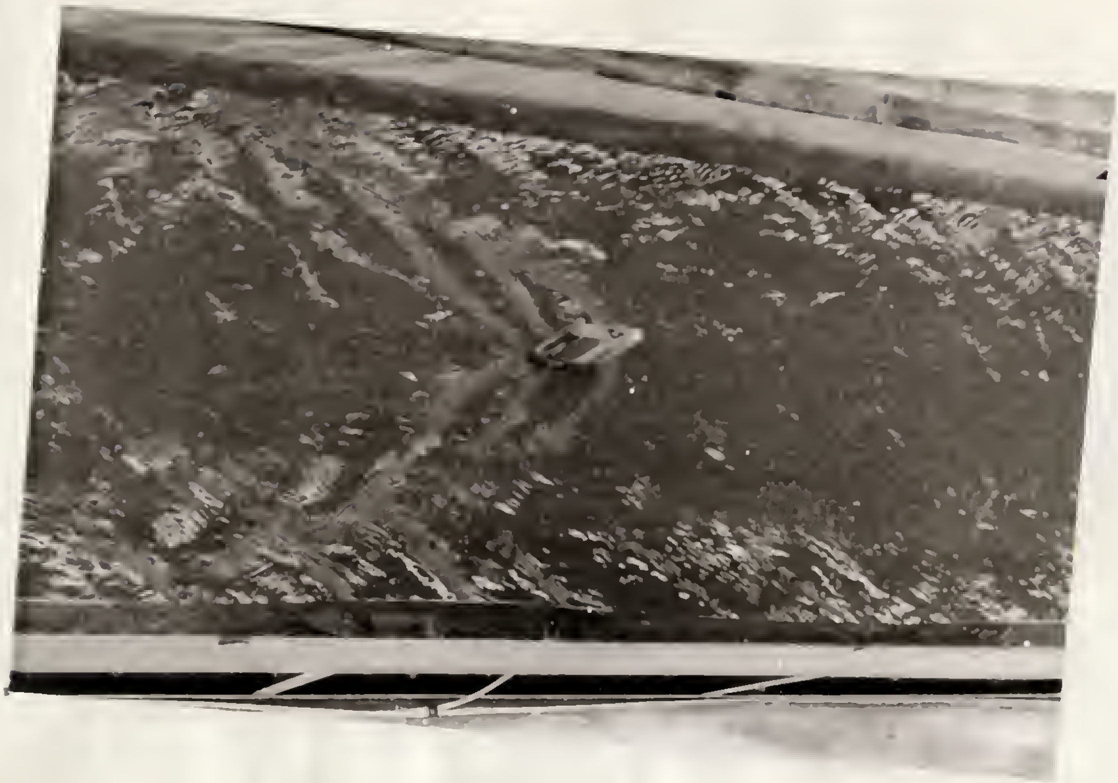
→ PLATE 27. FLOW CONTRACTION ARRANGEMENT (11.38%)(Exp.VI)

→ PLATE 28. FLOW CONTRACTION ARRANGEMENT (22.75%)(Exp.VI)

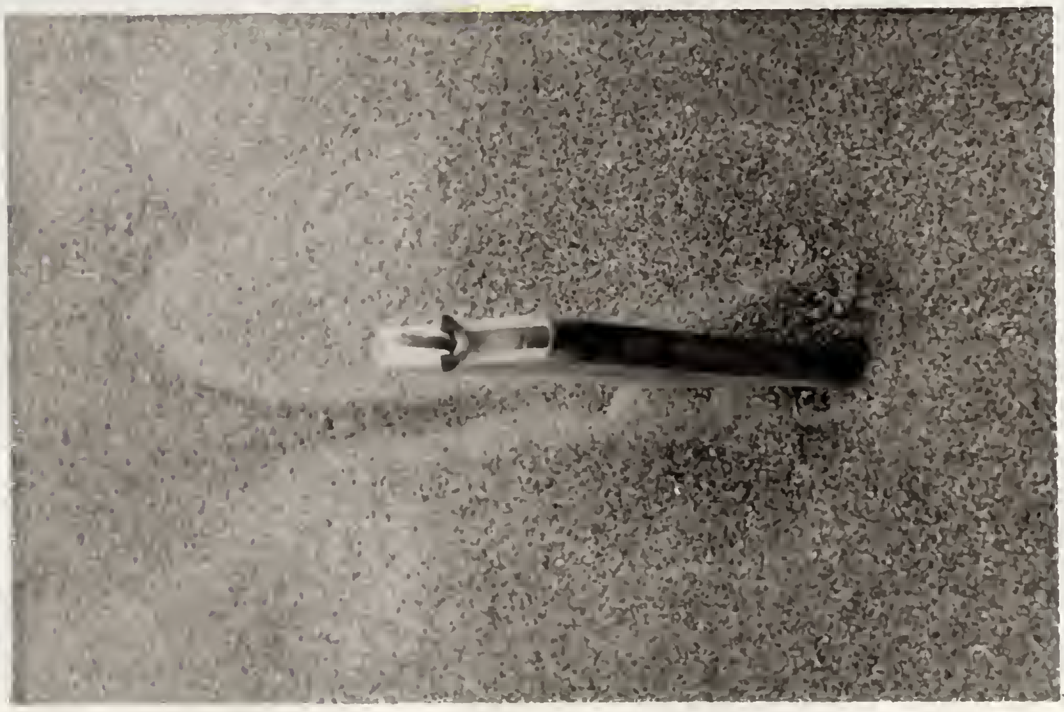


PLATES 29 and 30. FLOW AND SCOUR AROUND PLASTIC PIER (Exp. VII)

PLATES 31 and 32. DEBRIS ON PIER NOSES.



PLATES 33 and 34. FLOW AND SCOUR AROUND DEBRIS FREE PIER. (Exp VIII)



PLATES 35 and 36. FLOW AND SCOUR AROUND PIER WITH DEBRIS COLLECTED ON NOSE.(Exp.VIII)

PLATES 37 & 38. APRON I BEFORE
AND AFTER TESTING

PLATES 39 & 40. APRON A2 BEFORE
AND AFTER TESTING

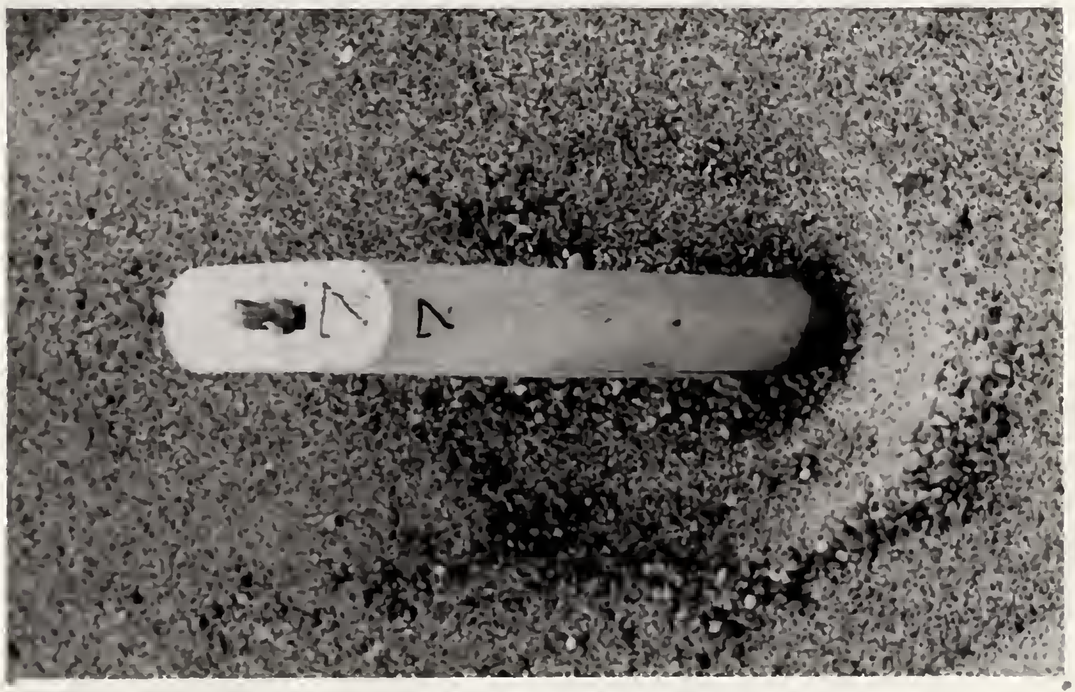
→ PLATES 41 & 42. APRON A 4 BEFORE AND AFTER TESTING

PLATES 43 and 44. APRON A7 BEFORE AND AFTER TESTING

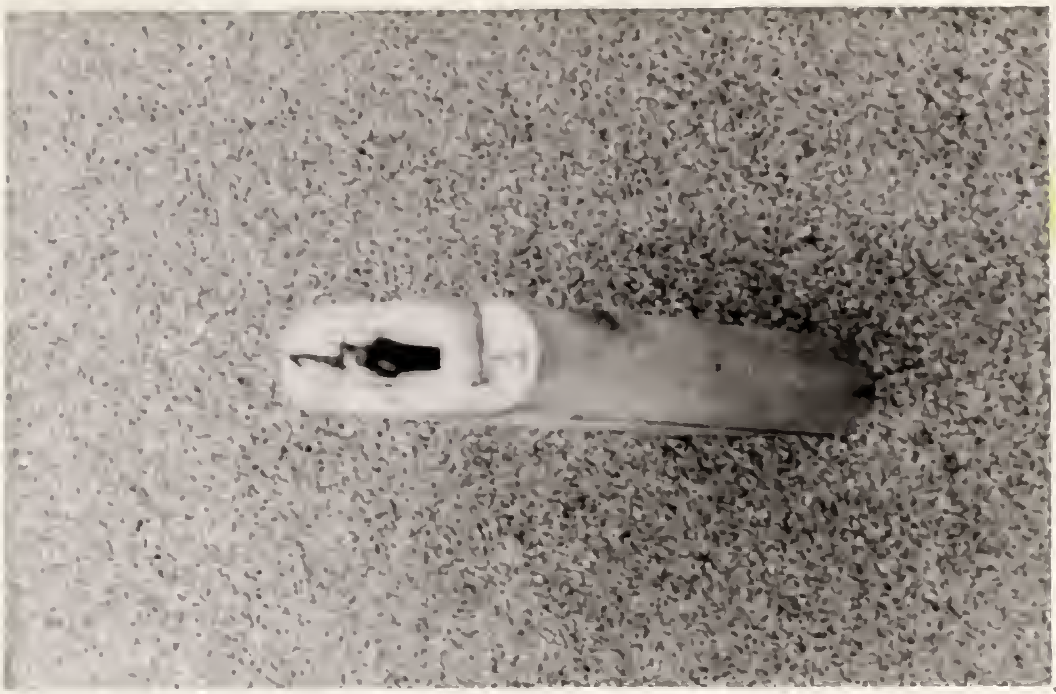
→ PLATES 45 and 46. APRON A9 BEFORE AND AFTER TESTING

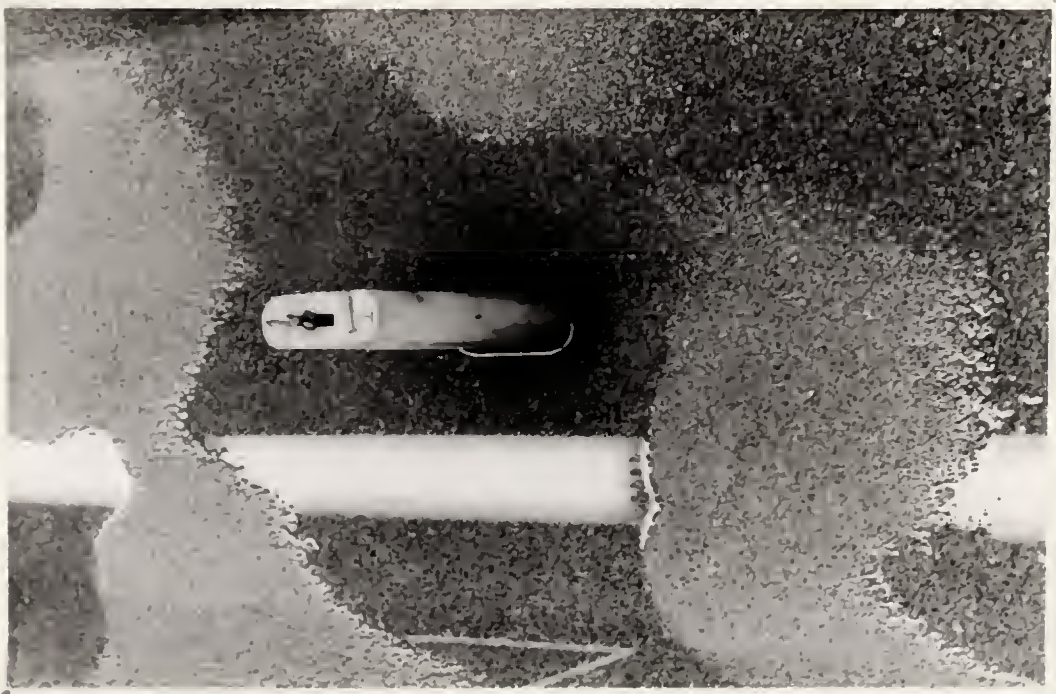
PLATES 47 and 48. APRON A10 BEFORE AND AFTER TESTING

PLATES 49 AND 50. APRON A11 BEFORE AND AFTER TESTING



PLATES 51 and 52. APRON A13 BEFORE AND AFTER TESTING





PLATES 53 and 54. INITIAL AND FINAL (SCOURED) BED AROUND UNPROTECTED PIER.